JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

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with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; P. H. BATES, ARTHUR R. LORD, J. C. PEARSON, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

Effort will be made to advise authors promptly of decisions as to availability for publication. Acceptance of a paper does not imply the acquiescence of critics (whose advice is sought prior to acceptance) or of the Institute in opinions which it expresses nor the acceptance of data or practice which it records. Institute authority attaches only to standards adopted as provided in the By-Laws. The publication of a committee report implies merely the cooperative effort of an appointed group within the Institute. Acceptance of a paper or report for publication does not commit the Program committee in reference to the annual convention.

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JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News Letter

MILWAUKEE—WEEK OF FEBRUARY 23

THE WEEK of February 23 next—Milwaukee, Wisconsin, Schroeder Hotel—time and place of a considerable group of meetings devoted to the study of the material concrete. These meetings include the six technical sessions of the 27th Annual Convention of the American Concrete Institute. A coincidence is the celebration of a quarter century of the organization's growth since the 2nd Annual Convention of its predecessor society, the National Association of Cement Users, in Milwaukee in January, 1906.

A large and active Milwaukee Convention Committee is holding frequent meetings and planning the local arrangements for the enjoyment and general success of the occasion. Kenneth H. Talbot, Director of Research of the National Equipment Corp. is Chairman, and Berry E. Brevik, Field Engineer, Portland Cement Association, is Secretary.

The Executive Committee of the local organization and the numerous sub-committees cooperating with the Institute on registration, trips, publicity, ladies entertainment, dinner, finance and membership, are listed.

Railroads in general are cooperating with reduced fares on the certificate plan, so that members of the Institute and dependent members of their families may make the round trip for a fare and a half, if 150 certificates are properly validated. There will be through sleepers from the East, directly into Milwaukee, of which see announcement elsewhere in these pages.

Convention registration will be in the foyer of the Grand Ballroom, Schroeder hotel, beginning Monday, February 23. That day the Concrete Masonry Association and the Wisconsin Concrete Products Association will hold joint meetings, continuing their convention on Tuesday. On Wednesday the Cast Stone Institute will meet,

and on Wednesday evening the Milwaukee Engineering Society and the Associated General Contractors of Milwaukee will join with the Institute in an informal dinner for which entertainment of high merit is being provided.

The convention of the Institute will begin Tuesday afternoon, February 24. The program of the various sessions is tentatively announced as follows:

FIRST SESSION—FEBRUARY 24, 2 P. M.

"Admixtures and Workability". Everyone recognizes the importance of workability as a quality in concrete for satisfactory placing but no generally accepted measure of it has been found. Closely identified with the necessity of a means for measuring workability is the matter of admixtures. Those who favor their use to improve workability, placeability, etc. are somewhat at a loss to prove their point in field practice for lack of an acceptable yard-stick. Discussion of these two allied subjects will be based on three papers by George A. Smith and Sanford Benham, by Inge Lyse and W. R. Johnson as published in the JOURNAL *Proceedings* pages, this month, and by Prof. G. M. Williams, University of Saskatchewan, on "Admixtures and Workability of Concrete," scheduled for the February JOURNAL.

Prof. S. C. Hollister, Purdue University, will present a paper on the effect on the properties of concrete in extracting water after placing and the means which have been used to reduce this surplus moisture.

C. H. Jumper, National Bureau of Standards, will report the results of an investigation at the Bureau to determine the value of intergal waterproofings in reducing the permeability of concrete, a portion of the investigation also having to do with applied coatings for the same purpose. Previous erroneous announcement was made that this paper would be by P. H. Bates.

W. R. Johnson, formerly of the Knoxville Power Co., Calderwood, Tenn. now of the Structural Engineering Dept., Purdue University, will describe unusual features in making and placing the concrete in the Calderwood tunnel construction.

SECOND SESSION—FEBRUARY 24, 8 P. M.

M. O. Withey, Professor of Mechanics, University of Wisconsin, will present a paper, "Some Long Time Tests of Concrete," scheduled for advance publication in the JOURNAL next month, in which he will summarize the results to date of two series of tests, one begun in 1910 to run for 50 years, the other in 1923 to run 100 years, to ascertain the effects of age and curing conditions on the strength of concrete.

"The Durability of Concrete" will be discussed in a symposium by members of Committee 801, Durability of Concrete, based on field surveys of many structures under a variety of conditions. F. R. McMillan, Secretary, R. B. Young, and others of the committee will participate.

Paul T. Norton, Jr., Professor of Industrial Engineering, Virginia Polytechnic Institute and Dan H. Pletta, Asst. Professor of Civil Engineering, University of South Dakota, will present a paper on "Permeability of Gravel Concrete" reporting the results of tests at the University of Wisconsin involving some 2000 specimens, 700 of them permeability specimens.

The report for Committee 108, prepared by Professors Raymond E. Davis

and G. E. Troxell on "Properties of Mass Concrete" will be presented for discussion. (See *Proceedings* pages, this JOURNAL.) This preliminary contribution from a new, large and important committee in the Institute summarizes available data and suggests an outline of a research program.

Herbert F. Faulkner, Assistant Engineer, City of Seattle, and R. R. Hubbard, Construction Superintendent, will present a paper on the "Design and Control of the Diablo Dam Concrete."

THIRD SESSION—FEBRUARY 25, 9:30 A. M.

W. D. M. Allan, of the Portland Cement Association, will present further results of a series of tests at the Portland Cement Association Research Laboratory on the shrinkage of concrete masonry, supplementing a paper previously presented to the Institute.

Benjamin Wilk, General Manager of the Standard Building Products Co. Detroit, throws new light on a matter of manufacturing practice which many had considered disposed of years ago. Batch mixers have very largely succeeded continuous mixers in the manufacture of concrete products. Mr. Wilk ran comparative tests of the efficiency of batch mixers and the new type of continuous mixers in his plant and got surprising results which re-opened the entire matter of continuous vs. batch mixers. This paper, to aid discussion at the convention, is scheduled for publication in the JOURNAL next month.

P. M. Woodworth, Chairman of Committee 708, will present for preliminary discussion a new "Recommended Practice for the Manufacture of Concrete Masonry Units" with a view to replacing the Institute's present recommended practice in this field.

Committee 704, Cast Stone Standards, Fred Weigel, Chairman, tentatively promises a report suggesting possible revision of the tentative specifications for cast stone.

Committee 709, W. D. M. Allan, Chairman, will propose for discussion "Specifications for Concrete Burial Vault Manufacture."

Committee 408 engaged on the preparation of a recommended practice for the use of color in concrete, will present proposed practice on one phase of this large problem, that of color in trowelled surfaces.

FOURTH SESSION—FEBRUARY 25, 2 P. M.

Interesting investigational work has been done in the last year in Wisconsin in studying sizes and combinations of aggregates for road work. Participating in this work are the U. S. Bureau of Public Roads, the Wisconsin State Highway Commission, and Sheboygan County. T. C. Thee, Highway Engineer, Bureau of Public Roads, will present "Field Practices in the Use of Multiple Sized Aggregates in Portland Cement Concrete Mixtures." A. L. Hambrecht, State Engineer, Wisconsin State Highway Commission, and George W. Langley, County Engineer of Sheboygan County will discuss Mr. Thee's contribution.

Prof. Emory D. Roberts, Marquette University, will make a brief report of studies of a "consistency indicator" as a means of control of the concrete mixes on road work in Wisconsin.

Committee 602, Frank I. Ginsberg, Author-Chairman, will present a report, "Design and Operation of Central Mixing Plants."

Committee 205, Grading of Aggregates, for Handling on the Job, I. E. Burks, Author-Chairman, will present

a preliminary report. The work of this committee is closely tied in, in some of its phases, with the work on roads mentioned above.

THE DINNER—FEBRUARY 25, 7 P. M.

The evening will be given over to the dinner, an informal occasion in which the Milwaukee Engineering Society and the Associated General Contractors of Milwaukee will join. President Duff A. Abrams will present a medal for the most meritorious paper presented to the 26th Annual Convention, and turn the occasion over to subjects entirely foreign to concrete, a matter in which the Milwaukee Committee is cooperating energetically.

FIFTH SESSION—FEBRUARY 26, 9:30 A. M.

W. A. Slater, Chairman, Committee 105, Reinforced Concrete Column Investigation, will present a progress report involving a considerable accumulation of data from tests now in progress for about a year. In this investigation the American Concrete Institute has the cooperation of Lehigh University and the University of Illinois in a work supported by a fund subscribed through the Institute. The investigation will cost more than $30,000. To make discussion of the results easier a progress report of this committee is scheduled for publication in the JOURNAL next month.

Committee 109 on Plastic Flow, Prof. Raymond E. Davis, Author-Chairman, will present a report summarizing available information on this subject of "time yield" in concrete.

Committee 308 will report through Albert Smith, Author-Chairman, an analysis of design considerations for hurricane exposures in concrete buildings.

Committee 502, Mixing and Placing Concrete in Buildings, Arthur R. Lord, Chairman, will move the adoption of its specifications, now tentative, as published last year and discussed in the JOURNAL since that time.

Committee 503, on Fabricating and Setting Reinforcing Steel, W. F. Zabriskie, Author-Chairman, Committee 504, Specifications for Ready Mixed Concrete, Miles N. Clair, Author-Chairman, and Committee 506, Concrete Specifications for the Small Job, Arthur R. Lord, Author-Chairman, will ask the convention to give tentative adoption to the specifications already published in the JOURNAL, on their assigned subjects.

SIXTH SESSION—FEBRUARY 26, 2 P. M.

The final session of the Institute convention will open with a brief business session and continue with a discussion of light weight concrete. The first two of the commonly used raw materials for making light weight concrete, cinders and burned clay aggregates, as reported on respectively by committees 203, Einar Christensen, Author-Chairman, and 204, George E. McIntyre, Author-Chairman, will be followed by the report of Committee 406, Use of Light Weight Concrete in Buildings, Frank A. Randall, Author-Chairman, presenting a complete design analysis for buildings of various heights and loadings with various weights of concrete, (50 lb. per cubic ft. up) and discussing the very remarkable effects of the light material in the design and the economy of the structure.

These three committees will open the way for a general discussion of

light weight concrete, both materials and uses, which is a matter of lively interest in many branches of the concrete field.

Arthur B. Morrill, Assistant Engineer Filtration, Detroit Department of Water Supply, will describe unusual features of design and construction of a 20-million gallon water reservoir.

Committee 607, Herbert Coffman, Author-Chairman, will report in summary, available information on practice in use of cement in bulk.

TRIPS

The Milwaukee Convention Committee will be prepared to arrange inspection trips for small groups on short notice Wednesday morning or afternoon or Thursday morning.

THE LADIES

Tuesday afternoon visiting ladies will be entertained at a tea; in the evening at a Theatre Party; Wednesday at a Bridge-Breakfast; in the evening they will join the men at the Dinner; Thursday they will have an opportunity to see how hosiery is made, or candy, in either of two nationally famous Milwaukee factories.

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THE HONOR ROLL

BRINGING new members into the fold is not in general an isolated activity of those members who excel in this direction. Look through the list on the Honor Roll and you can't help noticing that those successful in sponsoring new members have been meeting their membership responsibilities in technical and administrative effort in very energetic fashion.

The Honor Roll for 1931 has a month to go—closing date February 1.

W. D. M. Allan has sponsored 19 new members since February 1, 1930. Professor Davis is next with nine and F. R. McMillan close behind. There is a month in which to change these figures very materially.

W. D. M. Allan	19
Raymond E. Davis	9
F. R. McMillan	8
Miles N. Clair	5
D. A. Abrams	4
P. G. Gauthier	4
H. J. Gilkey	4
F. I. Ginsberg	4
Einar Christensen	3
J. C. Pearson	3
F. E. Richart	3
H. C. Turner	3
M. M. Upson	3

ALMOST

Charles S. Barr	2
Cloyd M. Chapman	2
Herbert Coffman	2
R. V. F. Eldridge	2
Louis F. Fossum	2
Herman Frauenfelder	2
Francisco Gomez-Perez	2
Master Builders Co.	2
J. H. Murdough	2
H. D. Oliver	2
Hale Sutherland	2
W. F. Way	2
R. B. Young	2

Eighty other members have one new member each to their credit.

Total 178

Institute members accompanied by their wives may be sure the visiting ladies at the Milwaukee Convention will have no opportunity for boredom while technical sessions are in progress. The Milwaukee Convention Committee is prepared to see that the ladies have an interesting program all their own.

IN CONSTANT REPAIR

(A Conversation with Dr. Sam'l Johnson)

Dr. Johnson: A man, Sir, should keep his Friendships in a state of Constant Repair.

The Author: And no less, Sir, his Information.

12—On Volume Changes

BY ARTHUR R. LORD

It is a well known fact, to which we have not yet given sufficient attention, that concrete is subject to change in dimensions as it hardens and, after hardening, as its moisture content or its temperature varies. This action it shares in common with every other building material in common use. We have a good beginning, but only a beginning as yet, toward the accumulation of test data covering this subject, most of which will be found reported in the *Proceedings* of the American Concrete Institute. On only one phase of it,—that related to reinforced concrete column design—have we had anything approaching an adequate discussion of the effect of these phenomena on the structural action and on the resultant stresses in reinforced concrete structures. Substantially all concrete structures are affected by volume changes, extending over their entire life. To poor concrete they may be fatal, to good concrete they are important only as they modify the stresses and structural action.

There is still a third volume change, which we have called "plastic flow" or "time yield," that occurs when concrete is subjected to considerable compressive stress for considerable periods of time. Recent tests by Gilkey indicate a similar volume or dimensional change under tensile and shearing stresses, as well. We have already much scattering data on plastic flow and a few engineers have printed their speculations as to its effect on the proper basis for design. Many years ago (I occasionally meet an engineer even today who still holds to this theory) there was wide discussion of a "theory of accommodation of stresses." This theory held that the distribution of the tension reinforcement between the center of span and the support was a matter of very much less concern than the elastic theory of flexure would indicate. If the steel at the top of a beam over the support was light, it would stretch accommodatingly when its stress reached the yield point and this would cause a greater deflection and stress at the center where the steel was heavier. The majority of engineers rejected this theory, on the basis that yield-point stresses in the reinforcement would open up large cracks in the concrete and precipitate a failure in diagonal tension. Still other engineers rejected it as improbable, saying that the plastic yielding with time would result in no appreciable redistribution of moments. Some of the advocates of this theory were in position to apply it to hundreds of buildings—and did so apply it. Generally speaking, they have not fallen down, have not exhibited the wide cracks and web failures that were predicted, are on the whole fairly satisfactory buildings. Is plastic flow the "good fellow" who saves the unprepared and the careless designer from a deserved poverty?

How do volume changes affect design considerations? We must remember in this connection that our accepted theory of flexure, our textbook standards, are all based on experience and reasoning involving speci-

mens in which our observations started after shrinkage was largely concluded, in which they were completed generally under conditions such as to preclude the other volume changes. Except in column design, they are totally omitted from our approved formulas. If low percentages of reinforcement in columns are found to be highly stressed (in compression) due to the shrinkage of the concrete (which is thrown into tension), is there any reason to believe that corresponding percentages of reinforcement in beams and slabs will not take similar initial compressive stresses? The few beam tests available on this subject indicate that such initial stresses do occur. Faber, speaking before the Institution of Civil Engineers in Great Britain, has discussed this subject and suggests that a modified value of n may be used to correct for the results of volume changes. Do other engineers who have thought on this subject agree with him on this simple method? Maney, in discussion before the Institute, has pointed out that compressive reinforcement in a beam will greatly reduce the plastic flow and increase the stiffness and utility of the beam under long continued loads. To what extent have these theories found actual application to design?

The subject is a difficult one to work through to any logical conclusion. Various concrete structures are variously exposed to changes in conditions such as result in volume changes in their concrete. If we are to modify n, what value shall we adopt in preference to the present standard series? Shall we assume in all designs, that the structure will be subjected to full design loading held in place over long periods of time, and so causing such great dimentional changes (without corresponding changes in actual stress) as have been observed in tests? We have all observed how differently the same temperature variations may affect similar structures, causing obvious distress in one and not even a hair crack in another. What shall we assume as a standard for purposes of design? We expect to further investigate this general matter in the tests now in hand in the investigation of reinforced concrete columns in cooperation between the Institute, Lehigh University and the University of Illinois. How shall we obtain the fundamental data necessary to extend this consideration to the effect, on the design of all kinds of concrete members, of all kinds of volume changes?

Concrete challenges the Engineer and keeps him alert as no other building material has ever done. It stimulates thought. It repays originality and application, by new economies. I recommend its cultivation by those who, like the green grocer in fiction, are troubled, when looking in their mirrors, by dead eyes, by faces like masks that never change expression as they talk. Concrete repays its votaries in mental keenness and growth. I invite you all to enjoy this invigorating recreation while assisting to keep our information in that state of constant repair which is the end and object of these articles.

Make your Milwaukee hotel reservations direct.

Institute members are promised the best of accommodations at reasonable rates. See notice elsewhere in these pages.

NEW MEMBERS

THIRTY-THREE applicants for membership in November have been approved by the Board of Direction as follows:

A. Angers, Box 114, Beaubarnois, Que., Canada.

Barzaghi, Arthur J., 13 East 37th St., New York, N. Y.

Balina, M. R., Copernics 2385, Buenos Aires, Argentina.

Blakeslee & Sons, Inc., C. W., 58 Waverly St., New Haven, Conn. (W. T. Gilbert)

Cast Concrete Products Assn., Ltd., The, 20 Dartmouth St. S. W. 1, London, England. (A. S. Windsor)

Chiao Tung University, Library, College of Civil Engineering, Tangshan, North China.

Concrete Marble Co., 5300 McKissock Ave., St. Louis, Mo. (G. C. Turner)

University of Detroit, Library, 6 Mile Road & Livernois, Detroit, Mich. (David P. Gilmore)

Devonald, Ira R., 1 Alan St., West Orange, N. J.

Diehl, Inc., George C., 577 Ellicott Sq. Buffalo, N. Y. (Roy P. Diehl)

Dominion Concrete Co., Ltd., The, Kemptville, Ont., Canada (W. G. Anderson)

Gallagher, John P., 16 Hawthorne St., Watertown, Mass.

Great Lakes Concrete Pipe Co. Inc., 973 Ellicott Sq., Buffalo, N. Y. (J. G. Miller, Treas.)

Harm, George O., 502 Reeves Bldg., W. Market St., Warren, Ohio.

Haynes, S. B., 511 Myrick Bldg., Lubbock, Texas.

Kamat, V. A., Shahpoor House, 114 Cawasji Patel St., Fort, Bombay, India.

Keatinge, Paul F., 208 S. LaSalle St., Chicago, Ill.

Krewatch, Wm., 13 S. Ann St., Lancaster, Pa.

Lucas, J. W., c/o Beauharnois Construction Co., Beauharnois, Que., Canada.

Macomber, C. Clark, 38 Chauncy St., Boston, Mass.

Matthews, L. H., Stone & Webster Eng. Corp., 49 Federal St., Boston, Mass.

Miller, N., c/o Beauharnois Construction Co., Box 50, Beauharnois, Que., Canada.

Molfino, Juan P., Lavalleja 1820, Montevideo, Uruguay.

Muir, Samuel, The Lehr Construction Co., St. Joseph, Mo.

Nonporite Pty. Ltd., 296 Burwood Rd., Hawthorne, Vict., Australia. (Norman Griffith)

Nova Scotia Power Commission, The, Halifax, Nova Scotia. (Harold S. Johnston)

Peters, Strange & Bradshaw, 617 Myrick Bldg., Lubbock, Texas. (N. L. Peters)

Reaney, Charles F., 5341 Goodwin St., Dallas, Texas.

Reinhold, W. J., Equitable Life Bldg., Queen St., Brisbane, Australia.

Valle, Juan Augustin, Hola de Mayo 1370, Direccion Nacional de Caminos Buenos Aires, Arg.

Woodstock Slag Corp., Southern Railway Bldg., Birmingham, Ala. (Geo. A. Mattison, Jr.)

Wurster Construction Co., 307 Architects Bldg., Los Angeles, Calif. (V. L. Marsh, V. P.)

Yeats, Alex J., 85 Farnham St., Belmont, Mass.

REINFORCED CONCRETE COLUMN TEST FUND

Cash, material donations, and services of assistants previously acknowledged (See December JOURNAL)	$24,159.53
Additional cash contribution since publication of December JOURNAL:	
Electric Tamper & Equipment Co., Ludington, Mich.	300.00
	$24,459.53

FLOW OF CONCRETE

A PAPER "Flow of Concrete under Sustained Stress," by Raymond E. Davis, Professor of Civil Engineering and Harmer E. Davis, Instructor in Civil Engineering, University of California, will present, first, a summary of the available literature upon the subject of plastic flow under compressive and flexure stresses and, second, a detailed account of a comprehensive series of tests to determine the flow of concrete under continuous compressive stress, which has been in progress at the University of California for more than four years.

The tests at California have been carried out under a wide variety of conditions and upon a large number of specimens. Among other things these tests have sought to determine the effect upon flow of: mineral character of aggregate, age at time of loading, water-cement ratio, richness of mixture, moisture conditions, etc. A series of tests has been made to determine the effect of alternately releasing and applying load for varying periods of time, from one day to six months. The flow has been found to be very considerably affected by the humidity of the surrounding atmosphere, by the age at the time of loading, by the mineral character of the aggregate, etc. It has been determined that under compressive stress, concrete when stored in dry air flows materially less than does the same concrete when stored under water. Many other interesting facts are disclosed. The paper is preliminary to the discussions by Committee 109 "Plastic Flow" of which Professor Davis is author-chairman.

THROUGH SLEEPING CARS TO MILWAUKEE

EASTERN members will be interested in the announcement of the Pennsylvania Railroad, by M. N. Luthi, division passenger agent, to run (in conjunction with the Chicago, Milwaukee, St. Paul & Pacific Railroad) through sleeping car service to Milwaukee, obviating a change of cars at Chicago.

Departure will be from New York at 11 a. m. Monday, February 23, arriving Chicago February 24, 8 a. m. thence to Milwaukee over C. M. St. P. & P. (Train No. 5), arriving Milwaukee 10:20 a. m. Philadelphia passengers may board the special cars at 12:55 p. m. Monday and Pittsburgh passengers at 10:30 p. m.

HOTEL ROOM RATES

THE HOTEL SCHROEDER was chosen as convention headquarters in Milwaukee. The management has agreed to hold a block of rooms up to three days before the Institute meeting to meet the advance reservations of those attending the convention. After February 20, the Hotel Schroeder and other Milwaukee hotels will do the best they can to meet the demands of all comers. On or before February 20,

your reservation at the Schroeder will be taken care of from the block reservations listed below. *Do not* send your reservations to the Secretary's office. Make your reservation direct to the hotel.

	One Person	Two Persons
125 Rooms, with bath	$3.50	$5.00
200 Rooms, with bath	4.00	6.00
50 Rooms, with bath	5.00	7.00
50 Rooms, with twin beds and bath for two		7.00

(Additional rooms with twin beds at $8.00, 9.00 and 10.00)

Do You Know Their Addresses?

FOLLOWING is a list of members who are on our return mail list. If you know the present address of any, will you send it to this office?

Domestic

Eades, Harry H., H. E. Marks Corp., Chatham & Phoenix Bank Bldg., Long Island City, N. Y.

Feast Cleland, Oneida, Tenn.

Hull, W. X., Hotel Adams, Phoenix, Ariz.

Kleinow, R. C., Johns-Manville Co., 230 N. Michigan Ave., Chicago, Ill.

Miles, F. D., Phoenix Utility Co., Ariel, Wash.

Peterson, Gordon S., 1042 College St., Toronto, Ont., Canada.

Ross, W. B., c/o Shell Petroleum Corp., Orange, Texas.

Smith, John Ward, Inspector, Pennsylvania Railroad, Philadelphia, Pa.

Foreign

Gamman, John, Building Surveyor's Dept., Town Hall, George St., Sydney, Australia.

Guruswamis, c/o Postmaster, Matunga, G. I. P., Bombay, India.

Hobbs, Merrill H., Apartado 682, Santo Domingo City, Republic Santo-Domingo.

Khan, Maqsood Ahmad, Vely Bazar House, No. 56, Meerut, U. P. India.

Tseng, T. W., Office of Director of Construction, Chengchow, Honan, China.

The members of the Milwaukee Convention Committee are all members of the Loyal Order of Milwaukee Hosts designated by an appropriate badge. Ask anyone wearing that badge.

Milwaukee—Week of February 23

The Concrete Masonry Association and the Wisconsin Concrete Products Association will hold a joint convention Monday and Tuesday, February 23 and 24.

The Cast Stone Institute will meet February 25.

The American Concrete Institute will have a get-together at Luncheon, 12 o'clock, noon, February 24; hold technical sessions at 2 and 8 p. m.; again February 25 at 9:30 a. m. and 2 p. m. and 27th annual dinner at 8 p. m. (joint affair with Milwaukee Engineering Society and Associated General Contractors of Milwaukee); again February 26 at 9:30 and 2 p. m. (See detailed tentative program, page 3.)

Registration begins February 23, 10 a. m.

Ride home from Milwaukee for half fare by asking for a "certificate" when you buy your going ticket at full fare. 150 such certificates validated at the convention headquarters will give you the half fare return rate.

Make your hotel reservations direct by addressing the hotel not later than February 20. See "Hotel Room Rates" elsewhere these pages.

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Properties of Mass Concrete

Report of Committee 108

A Summary of Existing Data

With A Suggested Program Of Investigation

BY RAYMOND E. DAVIS*, CHAIRMAN, AND G. E. TROXELL†

The hardening of portland cement concrete is a chemical process, and the hydration of the cement, being an exothermic reaction, is accompanied by a liberation of heat. When water is available, it appears that the rapidity of hydration depends upon the chemical composition of the cement, the fineness of the cement particles, and the temperature of the concrete of which the cement is a component part. Other things being equal, the finer the cement and the higher the temperature, the more rapidly does hardening take place. For most portland cements, hydration practically ceases for temperatures approaching the freezing point of water.

The total heat liberated by a given amount of cement from the time of beginning of hydration until hydration has been completed, can not readily be determined, but perhaps within ordinary limits of grinding may not differ as greatly with the chemical composition of the cement as is sometimes supposed, even though the rate of heat generation during the early stages of hardening may be very much greater for one cement than for another.

Any concrete, the temperature of which at the time of mixing is not above that of the surrounding medium, will undergo an increase in temperature, the heat generated by the hydration process being absorbed by the aggregates and uncombined water

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and gradually being dissipated by radiation until its temperature corresponds with that of the surrounding medium. At the beginning, temperatures increase rapidly, as does the hydration process. Then, as hydration is retarded, the rate of heat generation becomes less, until when rate of heat generation equals rate of heat dissipation, the maximum temperature is reached. Thereafter, as the rate of hydration gradually declines, the temperature decreases until that of the concrete reaches that of the surrounding medium.

It might be expected, then, that the maximum temperature attained by concrete due to chemical heat and also the time required to reach this maximum temperature would be influenced by (1) chemical composition of cement, (2) fineness of grinding, (3) character of aggregate, (4) richness of mix, (5) water-cement ratio, (6) initial temperature of the concrete, (7) temperature and character of the surrounding medium, and (8) the size and shape of the concrete mass. Thus it appears that there are at least eight factors, a variation in anyone of which may influence to greater or less degree the maximum temperature.

It is a matter of common experience that with our normal portland cements and normal mixes, the temperature rise in structures of thin section is too small to be of consequence. But it is also an observed fact that in concrete of massive proportions such as large dams, heavy foundations, thick arches, and the like, there may be a very considerable increase in temperature of the concrete subsequent to its placement, this increase sometimes amounting to more than 100 degrees F. and the interval during which this increase takes place sometimes being as long as a month or more. Because of the possible effect that these large changes in temperature may have, not only upon the properties of the material itself, but also upon the intensity and distribution of stresses and strains within the structural mass, it is of very great importance that the facts concerning the behavior of concrete in large masses should be known.

What follows is a summary of available information concerning the properties of mass concrete obtained for the most part from published writings, but to a considerable extent from unpublished data. An examination will show that beyond records of tem-

perature in a considerable number of structures, and some rather incomplete laboratory tests to correlate temperature and strength, very little is known of the properties of mass concrete. The observations of massive structures all indicate that the temperature rise is rapid and the temperature decline is gradual, in some cases extending over several years. While little quantitative data are available, it appears that in certain structures the decrease in temperature has been accomplished by cracking, indicating large tensile strains, and that in a general way the width and number of cracks increase with time. There are no data to show whether or not there is still further decrease in volume, after normal temperature has been attained, due to a gradual giving up of moisture within the mass which moisture is not employed in the hydration process; but it seems not unreasonable to suppose that mass concrete exposed to drying conditions may continue to give up moisture and gradually to shrink for many years after it has arrived at a state of practical temperature equilibrium, just as at a more rapid rate does the concrete in structures of thin section.

From such laboratory tests as are available it appears that high setting temperatures in mass concrete lead to high early compressive strengths but that ultimately the strength of concrete cured at normal temperatures is in excess of that which has undergone the more elevated temperatures within the mass. There is also evidence that when the temperatures are considerably above normal, the concrete within the mass may reach its maximum strength within a few days and thereafter may suffer a slight retrogression in strength, but tests are of too short a duration to demonstrate whether this retrogression is not perhaps later followed by a second increase in strength which may equal or exceed the amount of the retrogression. There is no information concerning the elastic or plastic properties of concrete subject to conditions obtained within the mass, nor concerning shrinkage strains.

EARLY TESTS

Results obtained from some investigations to determine the temperature rise due to the setting of cement follow:

Tests reported by J. B. Johnson on the rise in temperature at the center of a mass of portland cement concrete 11.5 ft. thick, show that the temperature rose 85.5 degrees F. in 4 days and reached its maximum increase of 93.6 degrees F. in 7 days, after which it fell off slowly. Data on the mix used are not stated.

The Watertown Arsenal made 12-in. cement cubes using 27 different brands. These specimens were in wooden forms with their tops exposed to the air. For these cubes, having an average initial temperature of 77 degrees F., the maximum temperature was 218 degrees F., showing a rise of 141 degrees in 13 hours. The lowest maximum temperature for any of the portland cements was 117 degrees F., a rise of 40 degrees.

The materials testing laboratory at Lehigh University made an 8-in. cube of 1:3 mortar, which rose from 70 to 160 degrees F. in 18 hours while the laboratory temperature was constant at 70 degrees F.

Tests on 6-in. neat cement cubes and on concrete as actually placed in construction work were made by the Universal Portland Cement Company at Buffington, Indiana. The 6-in. cubes rose from 80 degrees F. to 130 degrees F., a rise of 50 degrees in 8 hours, while the concrete rose from 70 to 110 degrees F., a rise of 40 degrees F. in 40 hours.

BOONTON DAM TESTS

Thaddeus Merriman,[1] one of the early investigators of temperatures developed in mass concrete, carried on tests during the construction of the Boonton dam in New Jersey in 1903. The total length of this dam was 3100 ft. and the maximum height of the full masonry section was 114 ft. Atlas portland cement was used exclusively in its construction. Although the downstream face was faced with ashlar masonry, the interior was of cyclopean concrete composed roughly of 50 per cent of concrete of a 1:3:6 mix and 50 per cent of large stones. All stone used was a syenitic gneiss and the sand was of glacial origin.

As work on the Boonton dam progressed, eleven thermophone coils were placed in the concrete at different elevations and at various distances from the two faces of the dam, but all in one central section. These thermophones were designed to be sensitive and correct to within 1 degree F.

[1]See Bibliography page 418 for all references.

The observed temperature record for thermophone No. 11, which was 8.5 ft. from either face and 11 ft. below the top of the dam, is typical of that for the other positions. The coil was set into the dam on June 24, 1904, and was immediately covered with about 12 in. of concrete which was at a temperature of 74.5 degrees F. Within the next 16 hours, it attained a temperature of about 90 degrees F. Seven days later, when the coil indicated a temperature of 77 degrees F., more concrete was placed above it, so that within 14 days, it was buried to a total depth of 11 ft. A second rise in temperature from 77 degrees to 93 degrees F. occurred in 13 days. This second temperature rise was largely due to the heat from the masonry placed above it, and consequently it can be said that the temperature of the masonry placed above coil No. 11, and insulated from it by 12 in. of concrete already set, must have reached a temperature higher than 93 degrees F.

The concrete in this structure was deposited in thin layers and therefore the heat was quickly dissipated. Merriman expresses the opinion that a temperature rise of 40 degrees F., could have been anticipated had the heat losses been reduced by depositing the concrete in deeper layers.

In the Boonton dam, about one year elapsed before the masonry returned to normal temperature. The Boonton dam records also indicate that minimum and maximum temperatures near the center of the dam due to seasonal temperature variations are reached approximately 30 days after the occurrence of the atmospheric minimum and maximum, respectively, and moreover, the "lag" is proportional in some degree to the distance from the nearest face of the dam.

R. H. Brown[2] reports tests conducted during the construction of canal walls for power development on the Kennebec River in 1909, at Madison, Maine. The concrete was a 1:3:5 mix, placed at a temperature of 70 degrees F. Recording thermometers placed inside a 3-in. pipe at the center of the thick walls (thickness not stated) registered a maximum temperature of 119 degrees F., reached after 8 days, when the depth of concrete over the thermometer was 26 ft. The temperature dropped to 98 degrees F. when the concrete was 20 days old. During these

observations, the outside air temperature averaged about 20 degrees F.

PANAMA CANAL LOCKS

Tests made to determine the setting temperatures in the Panama Canal Locks[3] show that a maximum temperature rise from 106 degrees to 130 degrees F. was attained 12 ft. from the surface. The maximum temperature occurred about 10 days after placing the concrete. Temperatures were observed by means of electric resistance thermometers imbedded in the side walls of the locks. The concrete was a 1:3:6 mix with some large plums.

DES MOINES ARCH BRIDGE

The concrete placed during construction of the Walnut Street arch bridge[4 and 6] in Des Moines, Iowa, was tested for rise in temperature while setting. The bridge consists of six 68-ft. reinforced concrete arches, the arch rings being 16 in. thick at the crown and 6 ft. thick at the spring line. Nine electric resistance thermometers were used in the tests. The maximum temperature rise occurred at the spring line, 6 in. from the intrados, where the 1:2:4 mix of concrete, placed continuously during night and day, reached a temperature of 108 degrees F., showing a rise of 32 degrees F. within 44 hours after placing. The mean daily air temperature was about 70 degrees F.

CEMENT PASTE TESTS BY BEALS

To determine the maximum temperature attained, and also to determine the rate of evolution of heat by cement pastes, Louis N. Beals, Jr.,[5] in 1913 reported tests upon small samples. Preliminary tests made by placing 900 g. of cement plus 225 g. of water in a can surrounded by a larger second can with paper insulation between, showed temperature rises varying from 14 degrees F. to as much as 45 degrees F. for the different brands tested.

As the heat due to the chemical reaction is evolved rather slowly and extends over a considerable period of time, the heat losses from the two cans described above were quite large. A second series of tests was therefore run using 1- to 2-lb. samples of cement in a thermos bottle with a secondary jacket to prevent heat losses.

The total calories liberated per gram of cement, corrected for radiation, were computed and plotted against time. It was found that different cements showed an appreciable difference in the rate of heat liberation and in the total heat liberated. It was also established that cements liberating only small quantities of heat, later proved to be unsound.

Beals' graphs show that if the radiation is taken into account, the evolution of heat continues over a period of more than 100 hours. Curves drawn to show the rate of heat generation in calories liberated per gram of cement per minute indicate that "there is a sudden evolution at mixing, then a sharp minimum —probably close to the time of initial set—followed by a longer maximum reaching a peak at about 8 hours, which may mark a more intrinsic property of the cement than the conventional final set. Another minimum and maximum follows, the significance of which may be developed by further investigation." The rate curves differed quite radically in some instances, as some cements indicated very high rates of heat evolution for short periods while others liberated the heat more slowly but for longer intervals. The total heat liberated by these two types of cement may or may not be the same.

W. A. Hoyt[7] states that the amount of heat generated depends upon the size or mass of the concrete, the kind of cement used, the consistency of the mixture, and also upon the richness of the concrete. A 1:2:4 mixture of medium consistency placed at 70 degrees F., in masses 12 in. thick or more, should develop a temperature of 120 to 150 degrees F. within 24 to 36 hours.

ARROWROCK DAM

Temperature changes in the concrete of the Arrowrock Dam, a gravity arch structure 330 ft. high, near Boise, Idaho, are reported by Paul and Mayhew.[8] The major portion of the concrete consisted of 1 part sand cement (45 per cent pulverized granite with 55 per cent portland cement by weight), 2½ parts sand, 5 parts of pebbles, and 2¾ parts of cobbles. A richer face mix, which varied in thickness, consisted of a 1:2:4:2½ mix. The cement in this face mix was composed of 76 per cent sand cement and 24 per cent portland cement by volume.

Ten electrical resistance thermometers were embedded in the concrete, all in one vertical plane, but at various elevations; some thermometers were close to the face of the dam, whereas others were deeply embedded in the dam. The temperature indicator, consisting of a modified Wheatstone bridge calibrated to read temperature units instead of ohms, could be read to 0.1 degree F. The Leeds and Northrup Co., the manufacturers of the coils and indicator, claimed an accuracy of the apparatus as a whole to 0.5 degree F.

Concreting at the location proposed for a thermometer was usually discontinued temporarily and the concrete allowed to set for a few hours before placing the coil. In some cases, concreting was not carried on continuously, directly above the thermometers. The delay in concreting preparatory to placing a coil, the shallow depth of cover of 1.5 to 2 ft. placed in most cases, the use of sand cement and the leanness of the mix used, all worked toward the development of lower maximum temperatures than would have resulted otherwise. As the investigators stated, however, the purpose of their observations was to determine the temperatures produced under average placement conditions for large dams where lean mixes are ordinarily placed in about 2-ft. layers with not more than one or possibly two layers being placed during one shift.

The results of the observations by Paul and Mayhew are presented in Table 1. As a result of these tests which show a maximum temperature rise of 36.7 degrees F., it was concluded that large bodies of concrete deposited at a temperature of about 60 degrees F. develop a temperature of from 90 to 95 degrees F. within a period of 30 days and maintain nearly that temperature for several days thereafter. Also, the effect of the "setting heat" in concrete 20 ft. or more from an exposed face is felt for several years after the concrete is placed. Concrete near the center of the mass of a large dam probably retains some of this "setting heat" for 5 years or more after placing.

Paul and Mayhew also state that the effect of the temperature of the air in counteracting the heat generated in the concrete by chemical action, is illustrated by the fact that all the comparatively low maximum temperatures recorded by several of the

TABLE 1—TEMPERATURE TESTS ON ARROWROCK DAM (PAUL AND MAYHEW)

Thermometer No.	Location of Thermometer	Days Required for Temp. Rise	Concrete Cover at Max. Temp.	Av. Air Temp. During Rise, °F.	Air Temp. at Max. Concrete Temp. °F.	Concrete Temp. °F.		
						Initial	Maximum	Change
1	76 ft. from W. face; 8.5 ft. W. of block placed 5 months previously; 60 ft. above bedrock.	27	15	71	60	66.5	94.0	27.5
2	3 ft. from W. face	5	8	55	57	55.0	74.5	19.5
3	20 ft. from W. face	45	28	44	26	49.5	86.2	36.7
4	58 ft. from E. face; 3.5 ft. below temporary top for 7 mo.	15	3	36	50	44.0	64.6	20.6
5	3.5 ft. from E. face; temporary top for 7 months only 3 ft. lower	5	4	48	48	43.0	69.8	26.8
6	31 ft. from E. face	31 101	20 80	73	77	61.5 61.5	93.7 96.2	32.2 34.7
7	2 ft. from W. face*	1	1.5	70	69	74.0	77.6	3.6
8	10 ft. from W. face*	17	30	58	58	75.0	91.6	16.6
9	1 ft. from E. face*	1	2	70	69	73.5	78.0	4.5
10	9.5 ft. from each face	5	10	48	51	47.0	80.0	33.0

*Thermometers 7, 8 and 9 directly above concrete 12 days old and under only 1.5 to 2 ft. of concrete cover for first two weeks.

thermometers occurred at points where the distance to an exposed face was comparatively small, and that all the high maximum temperatures recorded by a few of the thermometers occurred at points where the distance to an exposed face was considerable, and over which concrete to a considerable depth was placed during a comparatively short period. Therefore, it appears probable that the speed of placing has fully as much influence on the temperature of mass concrete as either the season of placing or the richness of the concrete. The duration of the rise in temperature, due primarily to chemical action, is undoubtedly

largely dependent on the speed of placing, season of the year, consistency, brand of cement, and quantity of cement per cubic yard of concrete.

In a discussion of the results reported for the Arrowrock Dam,[8] W. M. Smith calls attention to the fact that the proportion of portland cement in the loose materials by volume was only 5 per cent in the bulk of the masonry and 7 per cent in the facing. In the ordinary 1:3:6 concrete mixture, there is 10 per cent of cement by loose volume, and in a 1:2:4 mixture, generally used for reinforced concrete, the cement amounts to over 14 per cent. The statement is then made that had the cement component been higher in the concrete of the Arrowrock Dam, the setting temperatures would have been materially higher.

ASHOKAN RESERVOIR BRIDGE

The effect of a richer mix than that used in the Arrowrock dam is shown by tests on the spillway bridge at the Ashokan reservoir built by the Board of Water Supply of New York City. Several thermophones were placed at various points in the arch to record the temperature changes. Three were placed at the crown, five at a point about midway between the crown and the abutment, and three at the abutment. The arch was 3 ft. 4 in. thick at the crown, 4 ft. 8 in. at the midway point, and 6 ft. 3 in. at the abutment. The temperature rise at the crown was 20 degrees F., the maximum temperature being 88 degrees F. At the midway point, the rise varied from 27 degrees to 40 degrees F., according to the distance from the surface, the maximum temperature varying from 82 to 95 degrees F. At the abutment the rise varied from 30 to 50 degrees F., according to the distance from the surface, and the maximum temperatures varied from 95 to 115 degrees F. In every case, the maximum temperature was reached within 4 days.

In another test to determine the maximum setting temperature of concrete conducted during the construction of a shaft by the Board of Water Supply of New York City, it was found that a temperature of 136 degrees F. was developed.

KENSICO DAM

A very comprehensive series of tests to determine the temperatures developed in massive concrete structures due to the chem-

ical activity of the cement were conducted during the construction of the Kensico dam at Valhalla, N. Y., in 1913 and were reported by George T. Seabury in a discussion of the paper by Paul and Mayhew.[8]

Kensico dam is a gravity structure 1850 ft. long on top; its maximum height is 307 ft. and its volume is about 900,000 cu. yd. It is of cyclopean masonry, 27 per cent of the mass being of large stone; the remainder is of concrete mixed in the proportions of 1:3:6, Atlas and Alsen brands of cement being used. The concrete was handled rapidly in 2-yd. buckets, so that the elapsed time between mixing and placing was very short, usually not more than 5 or 6 minutes.

The portion of the dam chosen for the installation of the 47 sensitive electrical resistance thermometers is near the center. The thermometers were placed in two vertical planes; one near a contraction joint, and the other midway between two contraction joints, or in the center of a section. In each vertical plane the thermometers were placed in corresponding positions, and at various elevations; near the up-stream and down-stream faces, at equal distances from both faces, and at the middle line between these faces.

The central portion of the dam containing the thermometers was built comparatively rapidly. In the fall of 1913, a height of 35 ft. was placed, and in that height 15 thermometers were installed. During the following summer, the masonry was carried to the top—a distance of 165 ft.—in six months, and this part contains the remaining 32 thermometers.

At the Arrowrock dam it was the practice generally to place the thermometers in concrete which had already partly set and had been exposed for a long enough period to lose, by radiation into the air, a portion of the heat that would be normally developed. The thermometers, too, were covered to only a relatively small depth in many cases, permitting more radiation to go on, and so obscure the record of the total heat generated. On the other hand, at Kensico dam, the thermometers were plunged into the plastic concrete the moment it was placed, and were buried as deeply and as rapidly as possible in order to determine, if possible, the maximum temperature to which such a structure would be subjected under rapid placement conditions.

The results of the tests show that for a number of the locations, well away from the face of the dam, the rise in temperature was about 40 degrees F. and this seemed to be unaffected by the temperature at which the concrete was placed. This resulted, in the summer, in a maximum temperature well above 100 degrees. The highest temperature observed—and that which it is believed is probably the maximum in any part of the structure—was 118.5 degrees F., but this maximum temperature was not reached until long after the concrete had attained its final set. When undisturbed by other conditions, the maximum temperatures were reached in about 15 days—some points obtained their maximum temperatures in 5 days and others not for 95 days.

The increase in temperature due to the setting of the cement in the concrete began as soon as the latter was placed, and increased at the rate of about 1 degree F. per hour, for 4 or 5 hours, when the rate gradually increased to from 8 to 10 degrees per hour, for an hour or so, and then suddenly dropped to about 0.5 degrees F. per hour for a considerable time. The period of most rapid heat development corresponds with that of the final set of cement under ordinary conditions. Many observations indicated a rise in temperature of from 25 to 30 degrees F. after this period. As a consequence of this, the investigators raised the question: What is the effect on the strength of a large mass of concrete rapidly placed, if in its interior, a temperature of from 25 to 30 degrees F. is developed after the cement has ob-obtained its final set and the masonry its solidified form? Is the increase in size which results from this heating sufficient to develop disruptive strains in the fresh masonry beyond its early strength?

The opinion is expressed by Seabury that large bodies of concrete deposited rapidly develop a temperature a definite amount greater than the initial temperature of the aggregate, probably dependent almost entirely, if deposited rapidly enough to minimize radiation, on the quantity of cement to the unit of the resulting mass, and that this maximum temperature may be even 135 or 140 degrees F.

Meager evidence showing how the brand of cement affects the quantity of heat generated was ascertained, as there was one place in the Kensico dam where two thermometers were placed

about 5 ft. apart and under conditions which were thought to be identical, with one exception. After each had been covered by about 2 ft. of concrete made with Atlas cement, additional cover over one thermometer was made using the Atlas cement, while Alsen cement was used in the mix over the other thermometer. Within the first 6 hours after being placed, the thermometer under the Alsen cement did not rise as high as the one which was entirely surrounded by Atlas cement, and which had no Alsen near it, but, subsequently, they reached almost exactly the same temperature. The evidence is, then, that the brand of cement appears to make some difference at a certain stage, but not ultimately. However, these two thermometers were too close together to give reliable results unaffected by the heat from the other mix.

Seabury differs with the statement of Paul and Mayhew that "it appears probable that the speed of placing has fully as much influence on the maximum temperature of mass concrete as either the season of placing or the richness of the concrete." It was clearly demonstrated on the work at Kensico that, provided the initial heat acquired in setting be not dissipated through a slight amount of cover, the rise in temperature is a constant for any given mix, irrespective of the season of placing. It appears that the speed of placing has a great deal to do with the rise of temperature so far as reducing it is concerned; but, on the other hand, it seems that with a sufficient amount of cover, the rise will be a constant for any given mix. From this it follows that the season of placing, or rather the initial temperature of the aggregates and mixing water, has much to do with the maximum temperature reached.

The investigators at Kensico concluded that the rise in temperature due primarily to the chemical action, is undoubtedly largely dependent on the speed of placing, consistency, brand of cement, and quantity of cement per cubic yard of masonry but that these factors do not necessarily affect the duration of the rise in temperature as stated by Paul and Mayhew.

EAST CANYON CREEK DAM

Rather meager temperature observations have been reported for the East Canyon Creek dam,[9] built in 1915, near Morgan,

Utah. This is a true arch structure, 190 ft. high, having a thickness of 26 ft. at the base and 5 ft. at the top.

The proportions of the concrete used in the body of the dam were 1 part cement; 3 parts sand, with some screenings up to ¾-in.: and 5 parts broken stone of 3-in. maximum size. Red Devil brand of cement was used, and the cement content ran 4.75 sacks per cubic yard of concrete.

The concrete was deposited in 2-ft. layers, working back and forth. The aggregates were often frozen, but hot water added at the mixer raised the temperature of the fresh concrete so that it was just above the freezing point as it came from the mixer.

The rise in temperature due to chemical changes in setting prevented damage from frost when the concrete in the form was covered at night. Canvas spread over the forms and salamanders burning coke, aided in preventing surface freezing; while the interior temperatures of the mass soon increased sufficiently to remove any frost occurring on the surface, if not too great in quantity.

The removal of 1½-in. pipes which had been placed in the contraction joints afforded a very favorable means of testing the rise in temperature in the concrete while setting. Tests of temperature by lowering a thermometer into the holes, indicated in every case a considerable rise, a typical test showing a temperature of 70 degrees F. at a depth of 15 ft. when the air was 14 degrees F., a difference of 56 degrees F. from the air temperature or a difference of about 35 degrees F. from the initial temperature of the fresh concrete.

TESTS BY YOSHIDA AT UNIVERSITY OF ILLINOIS

While carrying on studies of the cooling of fresh concrete in freezing weather at the University of Illinois, Yoshida[10] also made tests to determine the rise in temperature during the setting of concrete and mortar.

The mixes used in these tests were made with Universal portland cement. The 0 to ¼-in. sand and the ¼ to 1½-in. pebbles came from pits at Attica, Indiana. The concrete or mortar to be tested was poured into a mold of 1½-in. pine lumber large enough for a 12-in. cube. Two thermocouples were embedded

to measure the temperature, one at the center of the cube and the other at 3 in. from the center. The mold was put in the center of a large wooden box and the space around all six surfaces of the mold was filled with sawdust about 8 in. thick, to prevent the cooling of the specimen.

The initial temperature of the fresh concrete or mortar was the same as that of the laboratory. The two thermocouples showed about the same temperature, so the temperature at the center was used in the preparation of Table 2. The total rise in temperature, the rate of increase, and the time interval before the maximum temperature was reached were all variable, depending upon the proportions of the mix and the amount of mixing water.

TABLE 2—RISE IN TEMPERATURE DURING SETTING OF CONCRETE AND MORTAR (YOSHIDA)

Mix	Water-Cement Ratio	Consistency	Initial Temp.	Max. Temp.	Temp. Rise	Hours to Develop Max. Temp.
Mortar			°F.	°F.	°F.	
Neat	0.38	Medium	68	184	116	15
1:1	0.48	Medium	68	126	58	24
1:2	0.60	Medium	68	108	40	32
1:3	0.85	Medium	68	86	18	24
Concrete						
1:1:2	0.72	Medium	72	89	17	29
1:2:4	0.81	Dry	73	85	12	30
1:2:4	1.01	Medium	75	87	12	30
1:2:4	1.26	Wet	71	83	12	26
1:3:6	1.29	Medium	71	82	11	29

EMIGRANT CREEK DAM

During construction of the Emigrant Creek dam,[13] holes about 2 in. in diameter extending from the downstream face to within 3 ft. of the upstream face were prepared at three different levels.

The thickness of the dam at the lower, middle and upper levels, was 15.5, 10.5 and 5.5 ft. respectively.

Long wooden rods of a diameter slightly smaller than the openings were then inserted in the holes after several maximum registering mercurial thermometers had been embedded in the

rods. Felt washers were used to prevent circulation of air in the holes. Daily readings taken when setting was progressing most rapidly indicated temperatures as high as 132 degrees F. Data on the concrete mix are not stated.

CALDERWOOD DAM

Tests of the temperatures in the Calderwood dam, a thin-section-arch overflow dam 230 ft. high, indicated a temperature of 148 degrees F. at the center. The concrete mix was 1:3:4.5 with a cement content of 4.5 sacks per cu. yd. The water-cement ratio was about 0.85, producing a slump of 1½ in.

WILSON DAM

A very complete program of temperature observations conducted during the construction of Wilson dam on the Tennessee river has been reported by John W. Hall.[16]

This structure is of the gravity type and has a total length of about 4300 ft. The spillway section, where the temperature tests were made, is about 94 ft. high and was built in 1923 and 1924. The concrete contained 5 sacks of cement per cubic yard and the maximum size of the aggregate was 3½ in. This concrete was mixed at central mixing plants, then dumped in 4-cu. yd. buckets of the bottom dump type and transported by railroad to the place of deposition. Each layer of concrete averaged about 6 ft. in depth.

Electric resistance thermometers were embedded in the concrete of one section of the dam, to determine the maximum temperature developed in the process of curing, and also to determine the length of time required for concrete to lose the effect of heat caused by chemical action of the cement. The resistance coils were placed in two vertical planes, one on the center line of the block, and the other 6 ft. from the expansion joint. Twenty-four thermometers were placed in each of these sections. The degree of sensitivity permitted the temperatures to be read to ¼ degree F.

As the thermometers were 1 ft. below the top surface of the layer being placed, the air temperature affected the thermometer until the next layer of concrete was placed, after which the thermometer recorded the temperature developed in the mass where it was located.

Some of the general points of interest from this series of observations are:

Thermometers 1 ft. from the face showed a rise from 50 to 83 degrees F. in 5 days, and then a fall to 43 degrees in 30 days, after which the temperature increased and decreased with the air temperature.

Thermometers 2 ft. from the face showed a rise of temperature from 64 to 82 degrees F. in 24 hours, and then a fall to 73 degrees in 3 days, at which time a new layer of concrete was placed and the temperature rose again to 86.5 degrees F. in 5 days and then gradually fell till the temperature reached 56.5 degrees F. at the end of 2 months.

Thermometers 9 ft. from the face, showed a rise in temperature from 63 to 79 degrees F. in 2 days and then a fall to 70.5 degrees in 3 days, when a new layer of concrete was placed, excluding the effect of the air temperature. The temperature then rose to 100 degrees in 11 days, after which it gradually decreased, reaching 76 degrees at the end of 3 months.

Thermometers 20 ft. from the face showed a rise of temperature from 64 to 84 degrees F. in 24 hours, and then a fall to 76 degrees in 3 days, when a new layer of concrete was placed. The temperature then rose gradually to 101 degrees in 11 days, after which it fell to 78 degrees in 5 months.

Thermometers 26 ft. from the surface showed a rise from 47 to 113 degrees in 24 days, and then a fall to 81 degrees F. in 7 months.

Information obtained from these temperature records shows that in this mass concrete, the temperature near the face increased slightly for a short period of time, approximately 4 or 5 days, and then fell slowly for as much as 30 days, after which the rise and fall of the temperature was directly affected by the air temperature prevailing. At greater depths in the mass, the temperature rose to as high as 113 degrees F. over a period of approximately 3 or 4 weeks, and then gradually decreased for a period of as much as 8 or 9 months.

It is interesting to note that this concrete was placed in the winter season when the air temperature, varying from 10 to 70 degrees F., but usually from 40 to 50 degrees F., would tend

to decrease the maximum reached by the mass, and yet this maximum was 113 degrees F. Also, it was noted that concrete near an expansion joint, even though in contact with adjoining mass concrete, does not increase to the temperature reached by the center of the mass, although this difference is apparently not over 3 or 4 degrees F. The information shown by this record is that the central portion of any mass concrete is sometimes in a state of expansion while the external surfaces are in a state of contraction, and explains the cracks appearing in the surface of concrete which do not penetrate to the interior of the mass.

Because of the high continuing temperatures in concrete of considerable volume, it was concluded by Hall that, if cracks in permanent work are to be avoided, deposits of concrete should be limited to 4 ft. for any one layer. Such deposits should be allowed to set and cool at least 5 days before succeeding courses are placed. Also, the greatest horizontal dimension for any given deposit should be 25 ft.

STEVENSON CREEK TEST DAM

In 1926, Slater[17] conducted an elaborate series of tests for Engineering Foundation upon the Stevenson Creek Test Dam in California in order to determine the distribution of stresses in arch dams. Temperatures of the concrete were observed from the time the concrete was placed until several months later.

This test dam is 60 ft. high and has a thickness of 2 ft. for the upper 30 ft. but below this the thickness increases until at the bottom it becomes 7.5 ft.

The concrete mix consisted of 1 part Colton cement, to 3 parts fine crushed granite aggregate varying from 0 to $\frac{3}{8}$ in., to 2 parts crushed granite varying from $\frac{3}{8}$ to $1\frac{1}{2}$ in., these proportions being by weight. The water-cement ratio varied from 0.87 to 1.03 and the consistency varied from a 2-in. to 7-in. slump.

In placing the concrete, a lift of 15 in. was deposited from end to end of the dam. The next lift was started at such time that at all points the concrete would have at least an hour in order for some set to take place between lifts. Ordinarily, one day's pouring consisted of four such lifts, or 5 ft. total. In general, about four days elapsed between pourings.

The setting temperatures of the concrete were measured by means of 140 electric resistance thermometers set at various points in the mass; some were close to the forms while others were deep seated in the concrete; some were at the middle vertical sections while others were at the abutments.

On the center line of the dam and 4 ft. above the base, where the thickness of concrete was 6 ft. 2 in., the temperature in the center of the mass rose from 73 degrees F. to a maximum of 116.5 degrees F. in about two days. From then on it gradually decreased, becoming constant at about 63 degrees F. after 3 weeks. At 2 in. from the face, the temperature rose from 72 degrees F. to a maximum of 95 degrees F. within one day, and from then on decreased at about the same rate as at the center, reaching a steady temperature of about 52 degrees F. after 2 weeks. The average air temperature was about 72 degrees F. when the maximum temperatures were reached. The minimum daily air temperatures averaged about 55 degrees F., and the maximum daily air temperatures averaged about 81 degrees F. during the first 3 weeks while the above mentioned tests were in progress.

MINNEAPOLIS BRIDGE

Jakkula[18] reports tests made in 1927 to determine the setting temperatures in the concrete piers of the Cedar Avenue bridge in Minneapolis. The concrete in this structure was made of portland cement, washed sand and trap rock, and the average field mix was 1:2.9:4.6. An inundator was used to measure the quantity of sand and to eliminate bulking. The average fineness moduli of the sand, trap-rock, and mix were 2.6, 7.8 and 5.6 respectively. The water-cement ratio was kept constant at 0.90.

Thermocouples were used for measuring the temperature of the concrete. Three of these were embedded in the central pier which was 8 ft. high, 50 ft. long and 25 ft. wide. The thermocouples were placed 4 ft. from the bottom on a line perpendicular to the center of the 50-ft. face, and at distances of 8.5 ft., 12.5 ft. and 16.5 ft. from that face.

Three thermocouples were also installed in each of the two shafts of the shore piers. These shafts were 16 ft. high, 14 ft. wide and 15 ft. long. The thermocouples were placed 3 ft. from

the bottom on a line perpendicular to the center of the 15 ft. side at distances of 1, 5 and 9 ft. from that side. All pourings were continuous at the rate of about 20 cu. yd. per hour.

Before any pouring was started, the forms were thoroughly warmed by salamanders. The sand, trap rock and water were heated by means of steam pipes. When each pour was finished, the entire section was covered with a tent-like arrangement of tarpaulins within which were placed salamanders heated with charcoal. Heating of the center pier was continued for 33 hours, of the downstream shaft of the shore pier for 54 hours, and of upstream shaft for 72 hours.

The records for the center pier show that the concrete was placed at 60 degrees F. when the surrounding air was at 40 degrees F. Between 8 and 38 hours after placing, the surrounding air averaged about 65 degrees F., and during this period the temperature of the concrete at all thermocouples rose 35 degrees to a maximum of 95 degrees F.

In the upstream shaft of the shore pier, the concrete was placed at a temperature of 45 degrees but fell to 40 degrees F., the temperature of the surrounding air, in 3 hours. Thereafter, the temperature of the air averaged about 65 degrees for 70 hours, at the end of which period the temperature 1 ft. from the face reached a maximum value of 92 degrees F. The temperature at 5 ft. from the surface continued to rise to a maximum value of 100 degrees F. at the end of 140 hours, at which time the air temperature was 30 degrees F.

In the downstream shaft of the shore pier, the concrete was placed at a temperature of 75 degrees but at the end of two hours fell to 69 degrees, when the air was at 38 degrees F. The air temperature gradually increased to 75 degrees F. at the end of 40 hours, and then fell to 66 degrees at the end of 54 hours. In the meantime, the concrete 1 ft. from the face rose to a temperature of 102 degrees F. at the end of 49 hours, and the temperature 5 ft. from the face increased to 105 degrees F. at the end of 54 hours.

PARDEE DAM

The Pardee dam, constructed in 1928, is a gravity type arch structure 350 ft. high above stream bed. The width at the base

of the dam is 244 ft., and the width at the top is 16 ft. The length of the dam is 1337 ft. and the volume of concrete is 615,000 cu. yd.

The concrete contained 4 sacks of Calaveras cement per cu. yd. and the aggregate in the mix, measured by volume, consisted of 30 per cent of 0 to $\frac{3}{8}$-in. sand, 28 per cent of $\frac{3}{8}$ to $1\frac{1}{2}$-in. gravel, and 42 per cent of $1\frac{1}{2}$ to 6-in. cobbles. Enough water was used to produce an average slump of 6 in.

In December 1928, when the dam had reached an elevation of 180 ft. above stream bed and 170 ft. below the top, five thermocouples were embedded in a line along the center of a block 75 ft. wide, located just south of the center line of the dam. The thermocouples were at 2 ft., 12 ft. and 55 ft. from the upstream and downstream faces, the latter point being at the center of the dam, as at this elevation the dam is 110 ft. thick. After placing the thermocouples, the concrete was poured in 5-ft. lifts at the rate of about one lift every 5 days.

The concrete was placed at a temperature of 71 degrees F. when the outside air was 39 degrees F. The temperature at the point 55 ft. from the two faces of the dam continued slowly to increase until it reached 102 degrees F., an increase of 31 degrees F., nearly seven months later when the air temperature was 77 degrees F. The average air temperature during this interval was 57 degrees F. The maximum rise in temperature 12 ft. from the face was 36 degrees F., occurring within 20 days, and the maximum rise 2 ft. from the face was 27 degrees, occurring within 12 days.

RESERVOIR WALL

In a discussion on high early strength concrete, H. V. McCall[19] states that in a series of tests upon a reservoir under construction during the winter where the concrete was placed at an initial temperature varying from 55 to 75 degrees F., the temperature developed within 3 days, in sections about 8 ft. thick, was approximately 60 degrees F. higher than when the concrete went into the forms.

PUTNAM COUNTY ARCH BRIDGE

Setting temperatures were determined during the construction of an arch bridge near Eatonton in Putnam County, Georgia[20].

This structure has an open spandrel arch with a span of 160 ft. and a rise of 46 ft.

The proportions of the concrete mix were 1:2:4 by volume, the fine aggregate under ¼ in. being a well-graded, clean quartz and the coarse aggregate, which varied from ¼ to 1½ in. in size, consisting of 50 per cent of quartz gravel and 50 per cent blast furnace slag. The time of mix was 2 to 5 minutes and the slump varied from 3½ to 4½ inches. The concrete was deposited in batches of about ⅓ cu. yd.

Temperatures were observed by means of electrical resistance thermometers embedded at different points within the mass. As typical of the temperature record, the concrete at the center of a quarter-point of an arch rib developed a maximum temperature of 131 degrees F. after 7 days, the initial temperature having been 77 degrees F. and the air temperature being 73 degrees F. at the time of occurrence of the maximum temperature in the concrete. The size of the arch rib at this point is not reported.

CHUTE A CARON DAM

Tests of the setting temperatures of the concrete entering certain auxiliary structures in connection with the Chute a Caron Dam in Quebec, Canada, are reported by I. E. Burks.[21]

The concrete in these structures had a water-cement ratio of 0.90 and the mix was approximately 1 part cement, to 2.3 parts of sand, to 4.8 parts of crushed stone by volume. The pit sand varied in fineness modulus from 1.9 to 3.6. The crushed stone was a feldspar obtained at the dam-site ranging in size from 2 to 6 in., and having a fineness modulus varying from 8.8 to 9.5. The concrete had no particles between ⅜ in. and 2 in. The compressive strength of the concrete was about 3,000 lb. per sq. in.

After a 2 to 3-minute mixing period, the concrete was hauled to the dam in 4-yd. buckets, and placed with the aid of vibratory tampers. The time interval between mixing and placing varied from 15 to 50 min.

In winter, the concrete materials were heated so that the concrete left the mixer at from 70 to 95 degrees F. and in some instances lost as much as 8 degrees before it was placed in the forms. After placing, the concrete was kept above 60 degrees F.

for 72 hours by covering with tarpaulins and heating with steam coils.

The temperatures within the concrete were observed by placing thermometers in wells made of ½ in. pipe from 3 to 8 ft. long. Concrete placed at a temperature of 78 degrees F. in Penstock No. 1, which is a mass 8 ft. thick, rose to 112 degrees F. at 2 in. from the exposed face at the end of 36 hours and the temperature at the center of the mass rose to 132 degrees at the end of 66 hours. The temperature of the air under the tarpaulin is not stated, but the outside air averaged about minus 5 degrees F.

In an obelisk pier, concrete placed at a temperature of 90 degrees rose to 157 degrees F. at a depth of 30 in. within 12 hours after placing. The air temperature outside the tarpaulin, averaged about 22 degrees F. during this interval.

TESTS AT THE UNIVERSITY OF CALIFORNIA

In 1929, under the direction of R. E. Davis and G. E. Troxell, several series of tests which have not been previously reported were carried on at the University of California, to determine not only the maximum temperature developed in mass concrete, but also to determine the effect of the high setting temperatures upon the compressive strengths at various ages.

For the first series of tests there were employed three different mixes of gravel concrete, as shown in the following table. The consistency of all mixes was kept constant at a 4-in. slump.

Mix No.	Water Gal. per Sack	Water-Cement Ratio	Approx. Mix	Proportions by Volume			
				Santa Cruz Cement	Sand	¼ to ¾ in. Pebbles	¾ to 1¼ in. Pebbles
1	4½	0.60	1:3	1	1.20	0.90	0.92
2	6	0.80	1:5	1	2.11	1.54	1.57
3	8	1.07	1:7	1	2.82	2.07	2.08

The sand was a well graded material varying from 0 to ¼ in. and having a fineness modulus of 3.03. The ¼ to ¾-in. pebbles and the ¾ to 1¼-in. pebbles had fineness moduli of 6.33 and 7.81 respectively.

TABLE 3—TEMPERATURES OF CUBES AND STRENGTHS OF CYLINDERS (DAVIS AND TROXELL)

Mix No.	Approx. Mix	Proportions by Volume	Gal. Wtr. Per Sk. Cement	Cylinders	Storage	Room Temp. °F.	Cube Temp. °F.			Hours to Max. Temp.	Compressive Strength, lb. per sq. in.					
							Initial	Max.	Rise		1-Day	3-Day	7-Day	28-Day	90-Day	1-Year
1	1:3	1:1.20:1.82	4.5	Inner	Cube	77	71	154	83	33	3110	3130	3720		4120[2]	4030
1	1:3	1:1.20:1.82	4.5	Outer	Cube	77	71	153	82	33	3080	3270	3710		3960[2]	3950
1	1:3	1:1.20:1.82	4.5	Control	Lab. Air[1]						995	2530	3200		5010[2]	
1	1:3	1:1.20:1.82	4.5	Control	Fog at 70°						1375	3000	3870		5750[2]	5220
2	1:5	1:2.11:3.11	6	Inner	Cube	68	60	115	55	94			3340	3840	3795	
2	1:5	1:2.11:3.11	6	Outer	Cube	68	60	115	55	94			3260	3780	3770	
2	1:5	1:2.11:3.11	6	Control	Fog[3]								2120	3620	4170	
2	1:5	1:2.11:3.11	6	Control	Fog at 70°[4]								2610	4230	4570	
3	1:7	1:2.82:4.15	8	Inner	Cube	67	59	101	42	97				2890	3240	3280
3	1:7	1:2.82:4.15	8	Outer	Cube	67	59	101	42	97				2890	3160	3360
3	1:7	1:2.82:4.15	8	Control	Fog[3]									2620	3420	
3	1:7	1:2.82:4.15	8	Control	Fog at 70°[4]									3180	3850	3830
3	1:7	1:2.82:4.15	8	Control	Air at 70°[5]											4050

[1]Specimens in air at about 60 degrees F. stored in paraffined cardboard molds with slip covers.
[2]After 7 days storage as indicated, cylinders were removed, coated with asphalt and stored at 70 degrees F. and 70% relative humidity.
[3]Specimens stored in fog at 70 degrees in paraffined molds without covers, for 28 days. Cylinders then sealed with tin covers, coated with asphalt and stored at 70 degrees F. and 70% relative humidity.
[4]Molds removed after 1 day.
[5]Specimens in cardboard molds sealed with tin covers, coated with asphalt and stored at 70°F. and 70% relative humidity.

For the 1:3 mix (using 4½ gal. of water) three cubes, each 3 ft. on a side, were made. Each cube was cast in a wooden form made of 2-in. lumber, placed inside of a second form of 1-in. lumber with a 6-in. space between, this space being filled with dry sawdust. While pouring the cube, twelve 6 x 12-in. cylinders of the same mix, in paraffined cardboard cylinders with metal covers at each end, were embedded in the cube as close to the center as possible—four in an inner circle and eight in an outer circle. Several of the cylinders had iron-constantin thermocouples embedded in them so that the temperatures developed in the concrete might be observed. Other thermocouples were placed at various positions in the cubes in order to observe the temperature differentials between the various parts of the mass. After completion of the cubes, their tops were covered with waterproof building paper and with 6 in. of dry sawdust. Thereafter, frequent observations were made of the temperatures within the mass and of the air in the room where the cubes were stored.

At the same time that the cylinders were embedded in each cube, 10 similar specimens were made to serve as control cylinders. Of these control cylinders, 4 were left uncovered in a fog room at 70 degrees F., while the remaining 6 were stored under normal atmospheric conditions at about 60 degrees F.

Four similar cubes, except for a 12-in. insulation of sawdust instead of 6 in., were made of the 1:5 mix (using 6 gal. water) and 3 were made of the 1:7 mix (using 8 gal. water). Some of the control cylinders for these cubes were removed from their forms after 24 hours and stored in the fog room at 70 degrees F. The remaining control cylinders were sealed in molds with tin covers, the entire mold coated with an asphalt emulsion, and the specimens then placed in the fog room at 70 degrees F.

The cubes were broken into fragments at ages varying from 1 day to 1 year, and the embedded 6 x 12-in. cylinders were tested in compression together with the corresponding control cylinders for the cube.

The critical data regarding setting temperatures of the cubes and the strengths of the 6 x 12-in. cylinders are given in Table 3.

From a study of the data of Table 3, it will be observed that the compressive strength of rich concrete cured in mass at high tem-

peratures is high, even at the early age of 1 day. It will also be noted that the gain in strength thereafter is slow. These data show that the use of rich mixes of portland cement concrete to obtain high early strength is feasible if provision is made to prevent the dissipation of the heat of chemical reaction for the first day or so. However, there is no advantage in retaining the chemical heat of rich mixes if high strength is not desired before 3 days, since at that age, concrete water-cured at normal temperatures would have equal strength. The superiority of concrete water-cured at normal temperatures over concrete cured in mass at high temperatures is plainly shown by the results of the 28 and 90-day and 1-year tests.

While observing the temperatures developed in the concrete cubes, it was noted that the temperature drop between the inner cylinders and a point 1 in. from the face of the cubes amounted to as much as 13 degrees F. for mix No. 1, whereas it was only 2 degrees F. for the other mixes. Part of the difference is undoubtedly due to the use of only 6 in. instead of 12 in. of sawdust for the richer mix, but some of the difference is due to the higher temperature attained by Mix No. 1.

A second series of tests was conducted using the two richer mixes of the first series. Three groups of cylinders were made for each mix. The first group were sealed in cans and subjected to the same temperature variations as the concrete in the 3 ft. cubes of series 1. The specimens of the second group were stored without forms in the fog room at 70 degrees F., while those of the third group were sealed in cans and kept at 70 degrees F.

For Mix No. 1, the specimens cured at the high temperatures gained strength quite rapidly for 3 days. A slight further rise was shown at 7 days, but some retrogression in strength was shown between 7 and 28 days. No further retrogression occurred at 90 days. The fog cured specimens, and those sealed and cured at 70 degrees F., gained in strength consistently during the 90-day period, the fog cured specimens indicating about 15 per cent higher strength than those sealed at 70 degrees F. The strength of the latter groups surpassed that of the first group at the ages of 28 and 90 days.

For Mix No. 2, the specimens cured at the high temperatures gained in strength for 28 days, and then showed a slight retrogression at 90 days. The other two groups gained strength more slowly at first, but attained nearly the same strength as the first group at 28 days. At 90 days, the strength of the fog cured specimens exceeded that of the cylinders cured at the higher temperatures.

In order to determine the effect of the brand of cement upon the maximum setting temperature and to ascertain the effect of this temperature upon the compressive strength, Davis and Troxell conducted a third series of tests upon 9 different brands of portland cement.

The proportions by weight of the concrete mix for these tests were 1:1.50:1.85, with a water-cement ratio of 0.67 by volume and a slump of 3 to 3½ in. The sand was well graded from 0 to ⅜ in. and had a fineness modulus of 3.71. The coarse aggregate was a crushed stone graded from No. 14 mesh to ½ in. and had a fineness modulus of 5.82.

Four 18-in. cubes were poured at one time. These 4 cubes were placed in a single large enclosure about 6 ft. square and 3½ ft. deep so that each 18-in. cube could be insulated with 12 in. of dry sawdust on all sides, and on top and bottom. Thermocouples were embedded at the center of each cube to observe the setting temperatures. The air in the room where the tests were in progress was kept constant at 70 degrees F.

The results of these temperature observations are shown in Table 4.

The compressive strengths of the concretes made from these 9 cements under various curing conditions were obtained by filling 45 tin cans 3 in. in diameter and 6 in. high, with each of the 9 mixes. Tin covers were fitted to the cans after filling. Of these 45 specimens made with each cement, 15 were stored in a room at 70 degrees F. and 50 per cent relative humidity, 15 were stored in fog at 70 degrees F., and 15 were stored in a high temperature chest in which the temperature was varied according to a time-temperature curve in close conformity with the setting temperatures of the mass concrete made of the various mixes. The 9 brands of cement were separated into three groups

TABLE 4—SETTING TEMPERATURES OF CONCRETE MIXES USING PORTLAND CEMENTS (DAVIS AND TROXELL)

Group	Brand	Temperatures, °F.			Age at Max. Temp., Hrs.
		Initial	Max.	Rise	
A	1	68	125	57	33
	2	65	127	62	32
	3	69	121	52	28
	4	68	120	52	40
B	5	64	128	64	25
	6	69	130	61	28
	7	66.5	132	65.5	32
C	8	69	133	64	30
	9	68.5	139	70.5	29

TABLE 5—COMPARISON OF COMPRESSIVE STRENGTHS AT VARIOUS AGES OF NORMAL TEMPERATURE CURE WITH HIGH TEMPERATURE CURE SIMULATING MASS CONCRETE CONDITIONS (DAVIS AND TROXELL)

Group	Brand of Cement	Fog Room at 70° F.			High Temperature		
		1-Day	3-Day	7-Day	1-Day	3-Day	7-Day
A	1	545	1355	2400	1050	2060	3080
	2	580	1925	2930	1755	2340	2645
	3	1190	3220	3930	3065	4090	4670
	4	530	1910	2990	1700	2380	2610
B	5	1060	1960	2985	2890	3485	3755
	6	865	2135	3290	2375	2765	3380
	7	845	2455	3550	2395	3020	3170
C	8	1310	2890	4020	2715	4100	4220
	9	1230	2900	3940	2640	3425	3450

Groups A, B, and C cured at maximum temperatures of 163, 173 and 180 degrees F. respectively.
Compressive strengths in lb. per sq. in.

—A, B and C—as shown in Table 4, and all specimens of each group were subjected to the same heat treatment.

Due to the fact that there was not a large difference in average maximum temperatures for the three groups, it was decided arbitrarily to raise the average maximum temperature of Group C and then to raise the maximum temperatures of the other groups

proportionately, but still to retain the same age in attaining the maximum temperature and the same shape of the time-temperature curves. This change in the maximum temperature was made to see if the differences, if any, in the cements could be better shown. A maximum temperature of 180 degrees F. was assumed for Group C. This established the maximum temperatures for Groups A and B at 163 and 173 degrees F. respectively.

Five cylinders of each mix were tested at the ages of 1, 2 and 7 days; the results of the tests being shown in Table 5.

From a study of the data, it will be observed that for all tests at the ages of 1 and 3 days, the cylinders cured at the high temperatures exhibited higher strengths than those cured at 70 degrees F. At 7 days, the strengths of the high-temperature cured cylinders exceeded the strengths of the cylinders stored at 70 degrees F. for 5 cements out of the 9 tested.

PORTLAND CEMENT ASSOCIATION TESTS

The results of a series of tests made in the laboratory of the Portland Cement Association (submitted by McMillan by letter), which have a bearing on the effect of temperature in the concrete mass on the compressive strength, are shown in Table 6. The specimens were all made from materials at room temperature, then placed in steel forms with cast-iron base and cover plates and submerged in water at various temperatures for the entire period of the test, the forms being removed after 24 to 48 hours. This storage permitted the specimens to acquire the temperature of the surrounding water which could be easily measured.

From a study of the data in Table 6, it is seen that at all ages tested, there is a noticeable increase in strength for all temperatures up to 212 degrees F., with the one exception of the 28-day strength for the highest temperature. For the 28-day tests, the concrete cured at 125 degrees F. showed a slightly greater strength than that cured at 212 degrees F.

CEMENT PASTE TESTS MADE AT U. S. BUREAU OF STANDARDS

Tests were made by P. H. Bates[22] at the U. S. Bureau of Standards upon 32 brands of portland cement to determine the maximum temperatures attained during the setting process and to ascertain the rate of temperature rise. The cements tested were

TABLE 6—EFFECT OF TEMPERATURE OF STORAGE ON THE COMPRESSIVE STRENGTH OF CONCRETE (McMILLAN)

Temperature of Storage, °F.	Compressive Strength, lb. per sq. in.			
	2-Day Storage	3-Day Storage	7-Day Storage	28-Day Storage
33	150	250	830	2480
42	280	410	1210	2690
65	740	1140	1950	3430
125	1680	2020	2710	3690
212	2000	2360	2930	3480

those selected for other investigations and reported by Committee C-1 of the American Society for Testing Materials (Proc. A. S. T. M., 1928, Part I, p. 233).

In conducting these tests 200 grams of cement were mixed with 67.5 grams of water by a high speed motor-driven stirrer for one minute, in a tinned can 2⅜-in. in diameter and 3 in. high. This was then placed in a thermos jar, closed and placed in a cabinet maintained at 70 degrees F. for 24 hours. The temperature was read by multiple junction base metal thermocouples and recorded by a sensitive potentiometer.

The time-temperature curves plotted from the observed data show an appreciable difference between the rates of temperature rise and the maximum temperatures attained, the latter values being shown in Table 7.

CEMENT PASTE TESTS BY MERRIMAN

Tht same cements used at the Bureau of Standards were also tested by Thaddeus Merriman, Chief Engineer of the New York Board of Water Supply[22]. The procedure followed was to mix 450 grams of cement with 112 ml. of water in a tinned can, 3 in. in diameter and 3 in. high. This was placed inside of a specially constructed calorimeter and the temperature of the mixture determined by the use of thermometers read at 15 minute intervals. The heat losses of the calorimeter were also determined at different temperatures and the temperatures read on the ther-

TABLE 7—TEMPERATURE RISE OF NEAT CEMENT PASTES

Cement No.	Results of Tests by Bates		Results of Tests by Merriman	
	Max. Temp. Rise Deg. F.	Hours to Develop Max. Temp.	Rise at 16 Hrs. Deg. F.	Rise at 24 Hrs. Deg. F.
1	77	11	196	198
2	31	20	124	140
3	47	10	158	158
4	33	15	138	149
5	63	13	170	171
6	45	13	176	184
7	81	12	171	174
8	39	14	172	174
9	30	17	159	159
10	47	12	138	147
11	35	15	172	180
12	41	15	135	151
13	52	11	180	187
14	41	13	156	176
15	50	17	169	171
16	32	13	171	179
17	41	16	182	185
18	34	12	160	172
19	30	13	142	154
20	18	14	100	118
21	68	13	174	177
22	70	13	205	208
23	45	15	159	162
24	88	10	153	153
25	52	11	141	149
26	36	15	147	166
27	48	12	155	156
28	37	11	121	127
29	37	16	162	171
30	45	14	159	159
31	45	13	142	151
32	32	16	129	138

mometer corrected according to the previously determined heat losses. The results are presented in Table 7, along with those obtained at the Bureau of Standards.

There is no assurance that the maximum temperatures developed in large masses of concrete made from the 32 cements of Table 7 would have the same relationships as for the neat cements in small amounts. Some cements which indicated a slow liberation of heat and a relatively low maximum temperature in the tests by Bates and Merriman, might have shown

much higher temperatures relative to those attained by the other cements, if used in large masses of concrete; and the reverse might be true for those cement pastes which exhibited high temperatures.

PROPOSED EXPERIMENTAL PROGRAM

From a review of what has gone before, it appears that very little is known concerning the properties of concrete in large masses. To determine these properties and the influence of various factors upon them obviously is not a simple matter, first by reason of the large number of variables involved, second because we are dealing with large volumes, and third because of the necessity of long-time studies.

It is believed that a comprehensive series of laboratory tests should be made to determine the effect upon: (a) rise in temperature, (b) compressive strength at various ages, (c) modulus of elasticity at various ages, (d) flow at various ages, (e) residual strains, (f) residual stresses, (g) permeability, (h) durability, and (i) volume changes as affected by: (1) chemical composition of cement, (2) fineness of cement, (3) admixtures, (4) richness of mix, (5) water-cement ratio, (6) initial temperature, (7) character of aggregate, and (8) size of mass.

Supplementing the laboratory tests there should be field investigations which may be properly divided into two parts: first, those which have to do with structures already built and in service; and, second, those concerning new work where conditions as regards materials and methods of construction may be closely controlled. It is clear that one of the very important functions of the committee is to promote the cooperation of those in charge of construction enterprises from which pertinent information may be derived. Acting as a guide, the committee should be in a position to do much in promoting field research and preventing duplication of effort.

The principal masses with which the committee is interested are dams, but heavy foundations, bridge piers, massive arches, large anchorages, and thick tunnel linings ought not to be overlooked, for, because of the use of rich mixes, the temperatures of these latter structures may considerably exceed those normally reached in dams.

It is evident from the summary presented earlier in this report that, aside from records of temperature, there has been published comparatively little quantitative data concerning the behavior of mass concrete structures. However, during the past few years attempts have been made to measure dimensional changes as well as temperatures in a number of dams in this country, using the electric telemeter for determining interior changes and the strain gage for observing surfaces changes. While it appears that many of the unpublished data thus obtained are not reliable, owing to the inexperience of observers or to the inaccuracies of measuring instruments, information from these sources should be brought together and the results summarized.

Of considerable value also would be the principal facts concerning the physical condition of dams and other mass structures which have been in service for some time (particularly the location and width of cracks) together with details regarding the concrete materials and the manner of mixing, placing and curing.

The question of whether or not there is a gradual shrinkage, over a long period of time, of dams located in arid regions due to a gradual dissipation of uncombined water is one of considerable importance. Strain-gage measurements should be begun on a considerable number of these structures when the facts concerning the quality of the concrete are known, and these measurements should be extended over several years.

Attention should be given to developing a test program calculated to produce reliable and fairly complete information concerning the behavior of mass concrete in actual structures, particularly dams, under the wide variety of conditions met in practice, beginning with the construction period and extending over at least ten years. In order best to coordinate this field work it is considered imperative that the program as a whole shall be carried out under the general direction of a single organization, with the cooperation of those in charge of the individual enterprises. Insofar as possible, the field tests should supplement the laboratory program, being calculated to furnish quantitative data regarding the effect of variations in materials and processes, and in size and shape of mass.

BIBLIOGRAPHY

1. Effect of Temperature Changes on Masonry; Discussion by Thaddeus Merriman; Trans. A. S. C. E., Vol. 61, (1908), p. 413.
2. Concrete Engineering, Oct. 1910.
3. Report of U. S. Isthmian Canal Commission, 1910, p. 122.
4. Determination of Internal Temperature Range in Concrete Arch Bridges, by Nichols and McCullough, Iowa State College, Engr. Exp. Sta. Bull. No. 30, Jan. 1913.
5. Thermal Activities of Portland Cement During the Period of Setting, by Louis N. Beals, Jr., Trans. A. S. T. M., Vol. 13, 1913, p. 720.
6. Concrete Bridges: Some Important Features in their Design, by W. M. Smith, Trans. A. S. C. E., Vol. 77, 1914, p. 738.
7. Heat Generated by Setting Concrete, by W. A. Hoyt, Concrete-Cement Age, Vol. 4, p. 288, 1914.
8. Temperature Changes in Mass Concrete by C. P. Paul and A. B. Mayhew, Trans. A. S. C. E., Vol. 79, 1915, p. 1225.
9. East Canyon Creek Dam, by A. F. Parker, Trans. A. S. C. E., Vol. 83, 1919, p. 590.
10. Studies of Cooling of Fresh Concrete in Freezing Weather, by T. Yoshida, Univ. of Illinois, Engr. Exp. Sta. Bull. No. 123, May 1921, p. 23.
11. Behavior of Mass Concrete Under Service Conditions by A. P. Davis, Proc. A. S. T. M. ,Vol. 23, Part 2, 1923, p. 177.
12. Use of Concrete for Dams, Tunnels, Culverts and Canals by A. P. Davis, Proc. American Concrete Institute, Vol. 20, 1924, p. 89.
13. Taking Concrete Dam Temperatures During Process of Setting, Engr. News Rec., Vol. 93, Dec. 11, 1924, p. 964.
14. Early High Temperatures in Alumina Cement Concretes, Engr. News Rec., Vol. 94, 1925, p. 320.
15. A Study of Temperatures in High-Alumina Cement and Methods of Curing, by H. S. Mattimore, Proc. A. S. T. M., Vol. 25, 1925, p. 192.
16. Testing of Wilson Dam Concrete by John W. Hall, Proc. Amer. Conc. Inst., Vol. 22, 1926, p. 497.
17. Stevenson Creek Arch Dam, by W. A. Slater, Proc. Amer. Conc. Inst., Vol. 24, 1928, p. 272. See also Proc. A. S. C. E., May 1928, Part 3, p. 94.
18. Setting Temperatures in Concrete Bridge Piers by A. A. Jakkula, Engr. News Rec. Vol. 102, p. 913, June 6, 1929.
19. High Early Strength Concrete. Discussion by H. C. McCall, Proc. Amer. Conc. Inst., Vol. 25, 1929, p. 331.
20. The Behavior of a Reinforced Concrete Arch During Construction, by S. B. Black, Proc. A. S. C. E., Nov. 1929, p. 2296.
21. Concreting Methods at the Chute a Caron Dam by I. E. Burks, Jour. Amer. Conc. Inst., Feb. 1930, p. 347.
22. Variations in Standard Portland Cement by P. H. Bates, Jour. Amer. Conc. Inst., Nov. 1929, p. 65.
23. Temperature Variations in Concrete Dams are Probable Cause of Unusual Stresses by A. Floris, Hydraulic Engineering, Dec. 1928, p. 736.

Readers are referred to the JOURNAL *for May, 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931.*

Introducing Two Papers on Slump Tests and Flow Table Determinations

Here we discover two pairs of experienced investigators, working independently, studying slump tests and flow table determinations as measures of that yet-to-be-satisfactorily-measured characteristic of concrete mixtures—workability. They have both established the important fact that relation of flow to slump is not constant—that the slump is relatively larger for the richer mixtures. Yet they arrive at opposite conclusions. Smith and Benham are for the flow table; Lyse and Johnson are for the slump test. Are there not other seekers after this important yard-stick of that characteristic, workability (which is now written into specifications, yet lacks definite clues to its identity) who, in the combined data reported in the two papers which follow, can discover a common ground —a reconciliation of interpretation, which to one authority is one thing and to the other something different? As a welcoming gesture toward such a reconciliation, the two papers, submitted to the Institute within five days of each other, are published together, and the discussion (May 1931 Journal) *will be combined. One member of the Publications Committee, while approving each paper in turn as a worthy contribution deserving publication, considers it an unavoidable imposition on the membership to publish two, where, under happier coordination, one might have sufficed. If in the ensuing discussion there should be even an approach to that coordination, the "imposition" may be justified.*—Editor.

A Study of the Flow-table and the Slump Test

BY GEORGE A. SMITH* AND SANFORD W. BENHAM†

The slump test, long used as a suitable and convenient means for measuring the consistency or wetness of concrete is recognized, nevertheless, as only a rough test for determining that condition or property. In the laboratory where more accurate control of the consistency is desirable the flow-table is frequently used and is generally looked upon as being the more satisfactory. There is always, however, considerable difficulty in attempting to compare flow-table data obtained by different experimenters because there is no standard method of operating the apparatus. The size, weight, height of drop, and speed of operation are not standardized. In order that wetness of concrete may be visualized from flow-table data it is imperative that the data be obtained under fixed conditions, as is the case with slump test data. Before such a condition can be brought about, information as to the functioning of the flow-table under different conditions is necessary.

In view of renewed interest in this subject the authors feel that the data here presented not only are opportune but point out characteristic tendencies which should be brought to the attention of others. It is believed that the results of this series of tests may be helpful torward establishing a standard method for performing the flow-table test.

PURPOSE AND SCOPE

It has been pointed out by several investigators that for a given condition of wetness as determined by the flow-table those mixes which contain the larger amounts of cement, and which are the more workable, quite consistently give the greater slumps. This condition has been observed by the authors for a long time

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and it has been felt that where the laboratory is endeavoring to simulate, in many ways, actual field conditions, either the same method of measuring the consistency should be used or a method which, though on a different basis, will give comparable results.

The tests with which this paper deals were made to determine the necessary flows to be used to give definite slumps so that more careful measurement of the wetness of the concretes could be made using the flow-table instead of the slump test. Though 15 ⅛-in. drops has been our usual measure for the work done in making the flow-table test, the program was extended to include both five and ten ½-in. drops because it was thought that the more vigorous action of the greater drop might give results paralleling the slump test more closely than did results with the ⅛-in. drops, and also because the ½-in. drop is used in a number of other laboratories.

The data were obtained on five mixes, from a 1:3:6 to a 1:1½:3, gaged to three slumps within the range 2 in. to 8 in. Consistency determinations were made using the slump test and the three different methods of making the flow-table test. The work was carried through all conditions both with and without Celite as an admixture. Incidental to the studies of consistency, the concrete was cast into cylinder specimens and tested in compression at 28 days.

In all, the study involved the making of 300 slump tests and 450 determinations of the flow and the testing of 300 compression specimens.

MATERIALS AND TEST METHODS

The cement used in this study was part of a 50-barrel lot of portland cement purchased from a local dealer. On delivery the cement was emptied on a concrete pavement, turned over twice and then stored in steel drums until used. Physical tests indicated that the cement met A. S. T. M. standard specifications for portland cement.

Air-dry, washed bank sand from Spring Lake, N. J., was used as fine aggregate. The sand was graded 0 to No. 4, had a fineness modulus of 2.90, and weighed 103.8 pounds per cubic foot. The sand as received was separated on the No. 8 and the No. 16 sieves.

Five sieve analysis were made on each fraction and the three sizes combined to give the desired fineness modulus when preparing individual batches.

The coarse aggregate consisted of air-dry, washed gravel from the same source as the sand. This was separated into four sizes and later recombined. The combined gravel was graded No. 4 to 1 in. (square openings), had a fineness modulus of 7.04, and weighed 105.2 pounds per cubic foot. Neither the fine nor coarse aggregate contained appreciable organic impurities and in an air-dry state showed practically zero absorption.

In all mixes containing an admixture commercial Celite for concrete was used.

PROPORTIONS

All mixes were proportioned on a dry rodded volume basis for the fine and coarse aggregate, using 94 lb. per cu. ft. as the unit weight of the cement. Throughout the study the ratio of sand to gravel was one to two by volume and the quantity of aggregate per batch was constant, viz., 0.10 cu. ft. (4.709 kg.) of sand and 0.20 cu. ft. (9.539 kg.) of gravel. To effect variations in the richness of the mix the quantity of cement alone was varied to obtain the following mixes:

Mix (by Volume)	Per Cent Cement	Cement Used (Kgs.)	Celite Content Lbs. Per Sack
1:1½:3	133	2.843	2.25
1:1⅔:3⅓	120	2.558	2.50
1:2:4	100	2.132	3.00
1:2½:5	80	1.706	3.75
1:3:6	67	1.421	4.50

Based on the cement content of the 1:2:4 mix these mixes represent cement contents varying from 67 to 133 per cent as indicated in the table above.

When Celite was used as an admixture the quantity used per batch was constant (68 grams). The resulting Celite content, in pounds per sack of cement, and the quantity of cement used per batch are also shown in the table above.

These quantities resulted in a quantity of concrete sufficient to fill the slump cone with a small excess.

MAKING TESTS

Without going into specific detail as to making the tests, suffice it to say that batches for one complete run of 30 mixes (a days' work) were prepared and standard practice followed in performing the remainder of the work. With regard to making the flow and slump tests some explanation, however, is necessary. Each of the five mixes was gaged to give what would appear to be 2-, 6- and 8-in. slumps. The first run was made determining the flow with 15 $\frac{1}{8}$-in. drops followed immediately by a slump test. The second run, on the day following, was made duplicating all conditions except for the manner of making the flow test, when five and ten $\frac{1}{2}$-in. drops were used instead of 15 $\frac{1}{8}$-in. drops. In obtaining the flow, using $\frac{1}{2}$-in. drops, the specimen was first jolted five times, measured and then given five additional jolts and the diameter (average of four) again determined. This procedure resulted in ten slump tests and five flow-tests for each of the three different methods of operating the flow-table. The work involved the making and testing of ten separate batches of concrete for each condition as to mix and wetness. The flow is expressed as the percentage increase in diameter of the specimen.

After the consistency test was completed the concrete was cast in a 6 by 12-in. cylinder mold, capped when one day old and cured in a fog room at about 70° F. for 28 days at which time it was tested.

It is felt that because of the various types of flow-tables used in different laboratories a partial description of the table used in these tests is warranted. The apparatus was purchased from the H. C. Cragg Mfg. Co., Washington, D. C., and is similar to that designed by the Bureau of Public Roads and to the table used at the Bureau of Standards. Specifically the top weighs 122.5 pounds, has a diameter of 29.5 in., and a thickness of 0.5 in. It is made of bronze reinforced with eight ribs 0.25 in. thick, tapering from $\frac{3}{8}$-in. near the circumference to a depth of 2.5 in. at the hub. The drop was changed from $\frac{1}{8}$ in. to $\frac{1}{2}$ in. by removing a washer

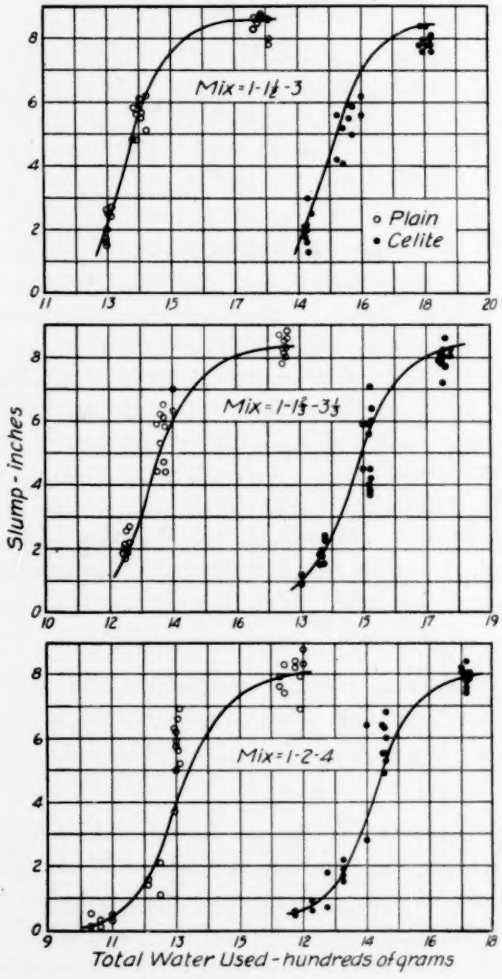

FIG. 1—RELATION BETWEEN SLUMP AND TOTAL WATER USED FOR VARIOUS MIXES

$\frac{3}{8}$ in. thick from the supporting shaft of the flow-table as used for the $\frac{1}{8}$-in. drop.

RESULTS OF TESTS

Rather than encumber this paper with tables of data, results of individual tests are plotted to show the grouping and variations in the data as well as the trends of the results. In all curves presented the open characters represent values for mixes without admixture and the solid characters those for concretes containing Celite. Figs. 1 and 2 show the slumps plotted against the actual weight of water used. Figs. 3 and 4 present similar data obtained for the three methods of using the flow-table.

The three methods of using the flow-table are again shown in Fig. 5 to indicate the change in water content for a given flow as the quantity of cement varies. Fig. 6 shows the change in flow for 15, $\frac{1}{8}$-in. drops and for the various mixes with change in water-cement ratio. All points indicated in Figs. 5 and 6 have been taken directly from the curves indicated in Figs. 3 and 4.

Fig. 7 shows the relation between the different methods of operating the flow-table based on data from all mixes.

Fig. 8 has been prepared to show the relation that existed between the slumps and the flows obtained when 15, $\frac{1}{8}$-in. drops were used.

Results obtained from the compression tests are shown in Fig. 9 where the average strengths from ten tests for each condition are plotted against the average water-cement ratio.

In Figs. 1 and 2 it may seem that part of the data in the region of intermediate slumps has been largely disregarded when drawing the curves. Such, however, is not the case. When experimental data are obtained using systematic variations in conditions the resulting trends obtained are usually systematic in their arrangement. It would, therefore, be natural to suppose that the relations of slump to flow would show the same or parallel trends when varying the mix. See Fig. 8.

In order to obtain the systematic arrangement of the curves shown in this figure it was necessary to make some adjustment of the water-slump curves. Since the trends of the flow data would permit little or no adjustment most of the adjusting was confined to the curves of water versus slump. Therefore, the

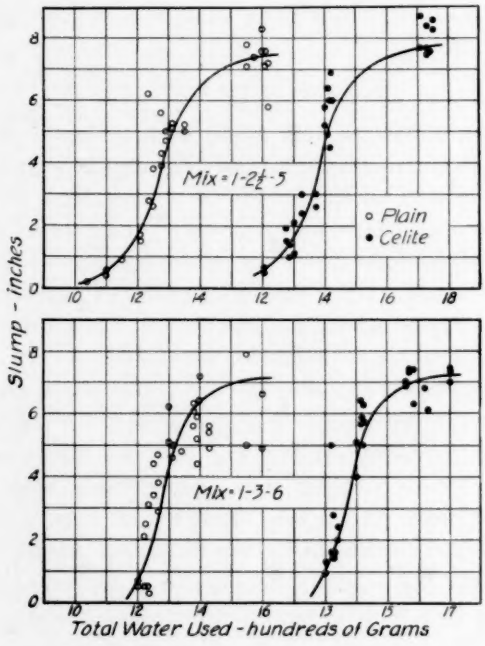

FIG. 2—RELATION BETWEEN SLUMP AND TOTAL WATER USED FOR VARIOUS MIXES

position and shape of a single water-slump curve, shown for any given mix, are governed not only by the data obtained for that mix but are influenced by the data obtained for all other mixes.

DISCUSSION OF RESULTS

The results obtained from the slump test (Figs. 1 and 2), though self-explanatory in many ways, call for some comment. It will be noted that the general shape of the curves is approximately that of a reversed curve with the point of inflection in the region of a 4-to 5-in. slump and limiting tangents at zero, probably, and in the region of 8 in. The curves are about the same shape as those shown by McMillan and Johnson[1] for similar mixes. For mortars and highly sanded mixes it is not improbable that the 8-in. upper limit may be increased as has been shown.[1]

[1] "Further studies of the Water-Cement Ratio Strength Relation in Concrete"—Report of the Director of Research, Portland Cement Association, November 1928, p. 16.

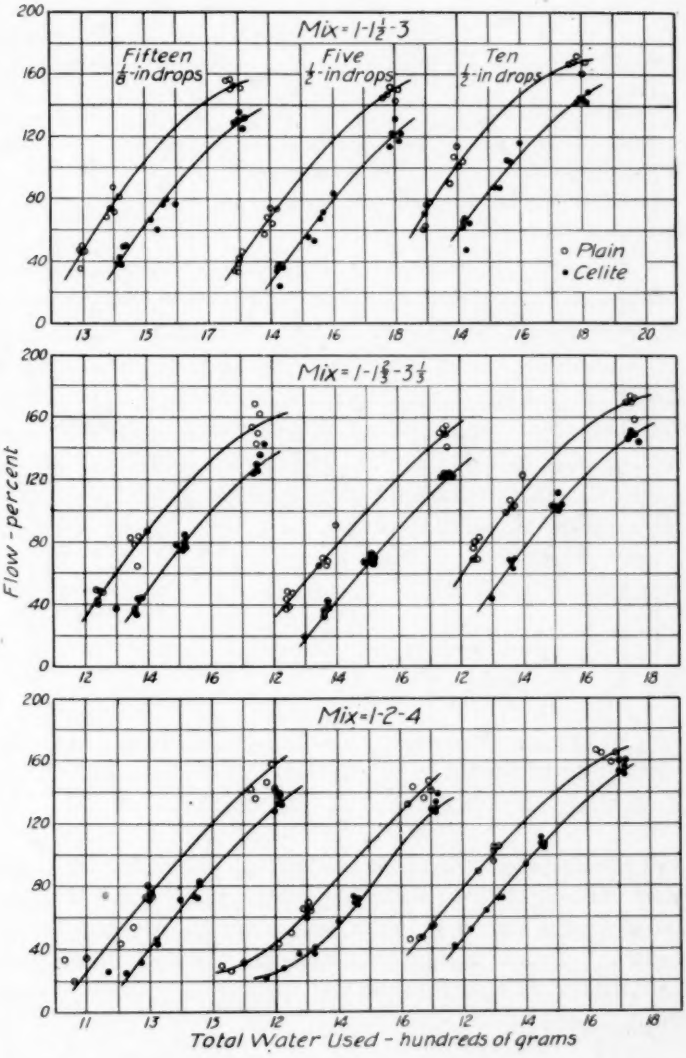

FIG. 3—RELATION BETWEEN FLOW AND TOTAL WATER USED FOR THREE METHODS OF OPERATING THE FLOW-TABLE

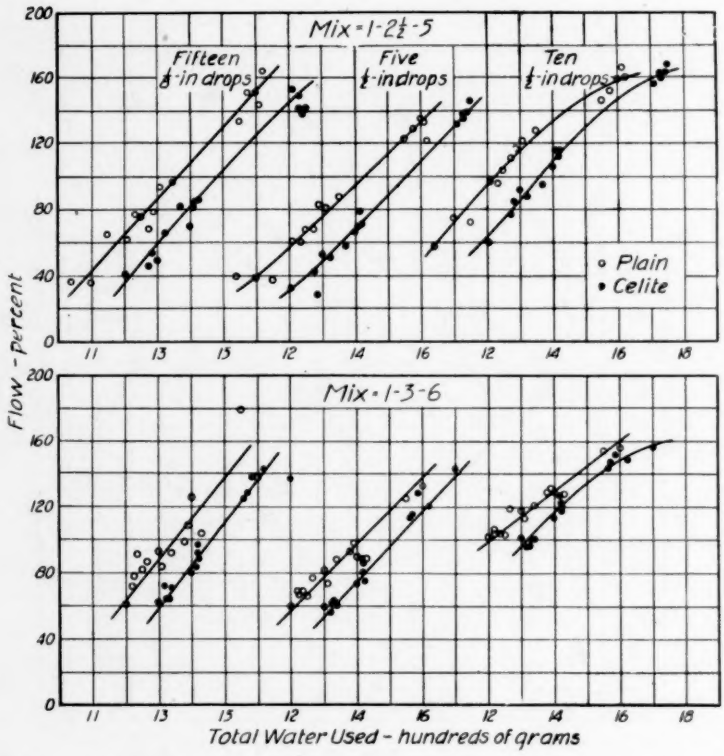

FIG. 4—RELATION BETWEEN FLOW AND TOTAL WATER USED FOR THREE METHODS OF OPERATING THE FLOW-TABLE

At both the low and high limits the test is less sensitive to change in water content than it is in the central range, if the condition indicated by the curves is taken as a criterion of the sensitivity of the test. In the range of intermediate water contents the individual data indicate that the test appears to function in a very uncertain manner.

During the making of the tests four characteristic conditions as to general shape of the resulting slump specimen were observed.

1. Specimen slumped only small amount and retained somewhat the same shape as the molded sample.

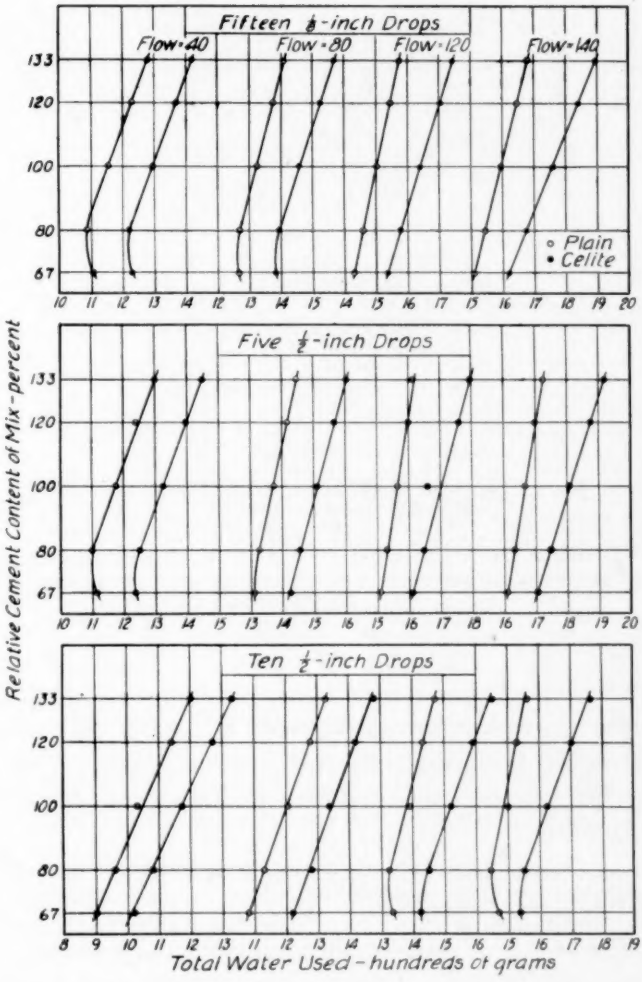

FIG. 5—RELATION BETWEEN THE RELATIVE CEMENT CONTENT OF MIX AND THE QUANTITY OF WATER REQUIRED FOR GIVEN FLOWS. CEMENT CONTENT OF MIXES IS BASED ON THAT OF A 1-2-4 MIX AS 100 PER CENT

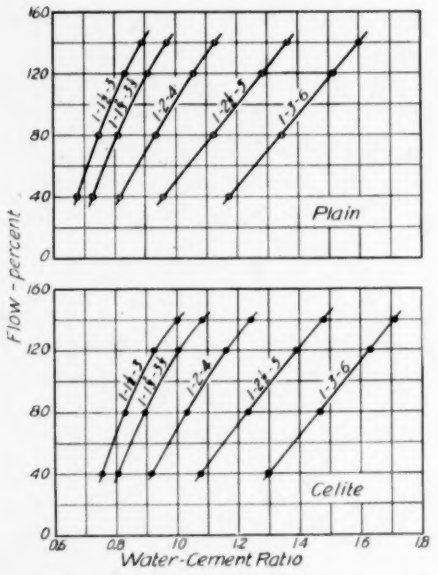

FIG. 6—RELATION BETWEEN FLOW AND WATER-CEMENT RATIO USING FIFTEEN 1/8-INCH DROPS OF THE FLOW-TABLE

2. Specimen slumped several inches but held together, bulging out uniformly as the slump took place.

3. A portion of the specimen stood up on one side and the remainder fell away. Not a usual condition and always checked by a second trial.

4. Top slushed down leaving a mound, generally of compacted aggregates, in the center.

In the region of 1- to 2-in. slumps, where the specimen conformed to Condition No. 1 above, the variation in slump for a given water content was relatively less than in other regions and in this region the slump test would appear to give some consistent indication of wetness.

In the upper region, 7- or 8-in. slumps, after a definite quantity of water had been used no change in the slump was produced by

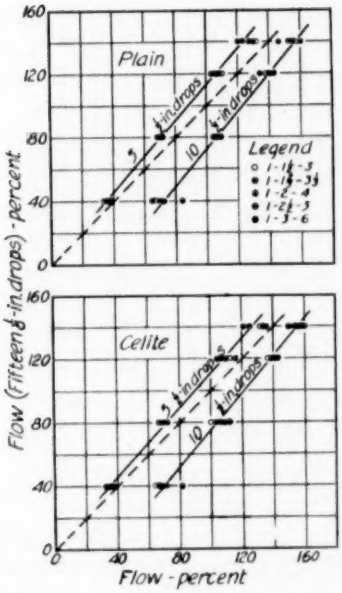

FIG. 7—COMPARISON OF DIFFERENT METHODS OF OPERATING THE FLOW-TABLE

increasing the water. In this case the test was characteristically that indicated by Condition No. 4. Due to segregation and compacting of the coarse aggregate a cone of approximately constant height was left as the upper part of the sample flowed down on all sides.

In the intermediate region of wetness (slump approximately 4-to 6-in.) the slump test was at its worst as is indicated by the wide range of slumps that were obtained with a constant quantity of water. Though the results generally showed somewhat better agreement in the richer and more workable mixes it was only in the richest mix considered that reasonable checks were obtained in this region. When trying for 6-in. slumps it was observed that the specimens assumed different shapes as represented by Conditions Nos. 2 and 3 or 4. That is, there appeared to be no con-

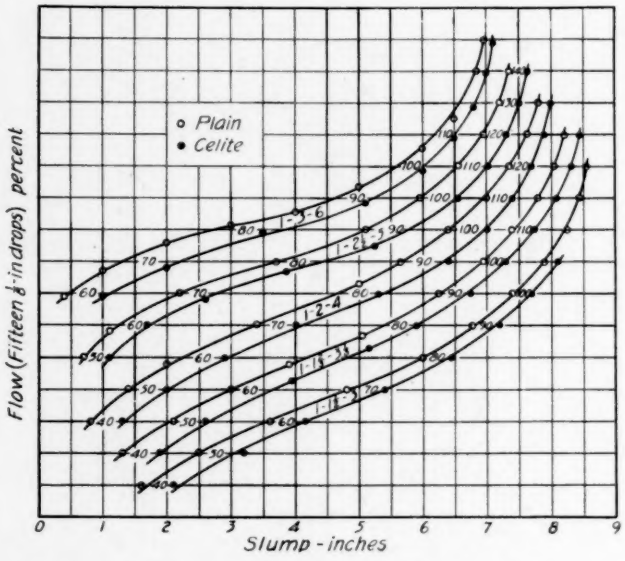

FIG. 8—RELATION BETWEEN FLOW AND SLUMP

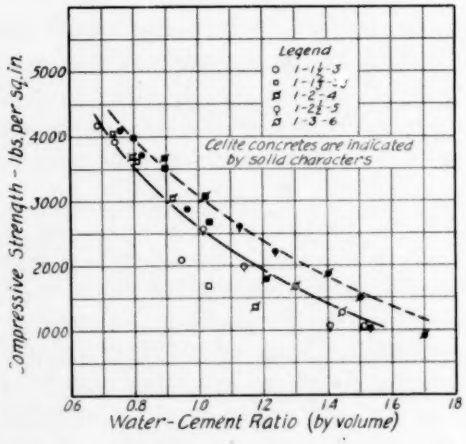

FIG. 9—RELATION BETWEEN WATER-CEMENT RATIO AND COMPRESSIVE STRENGTH AT 28 DAYS

sistent performance as was generally noted in the 2-in. or the 8-in. slumps. If Condition No. 2 obtained the slump would be small. If No. 4 obtained the slump would be large. If No. 3 obtained the slump might be any amount in between the slumps given by Conditions Nos. 2 and 4. The fact that each curve shows a point of inflection would appear to indicate that in this region the test was ceasing to function in one manner and beginning to function in another. This appears to be consistent since for moderate slumps, 4 in. or less, the test is usually a true slump or a settling of the mass as a whole if we omit the lean, harsh mixes which in this discussion have been given less weight than have the plastic workable mixes. Beyond the true slump condition the wetness is such that the ability of the paste to hold the mass together is overcome and the mix acts more as a semi-fluid mass.

Individual results from the flow tests (Figs. 3 and 4) show consistent conformity to average curves, unusually so when compared with the results obtained with the slump cone. An exception to this is shown in Fig. 4 for the 1:3:6 mix. Here, as in the slump tests, it is believed that lack of quantity and quality of the cement paste resulted in extreme harshness and segregation which are always undesirable and certain to give erratic results. However, even in the lean mixes, the flow-table data are in better agreement than are the slump data.

Within the range that may be classed as plastic concrete the flow is practically a straight line function of the quantity of water used. This holds quite generally regardless of the mix and regardless of which method is used in making the flow test. In this connection it should be kept in mind that the volume of aggregate was constant and that different conditions of richness were obtained by varying only the quantity of cement. In other words the volume of concrete resulting from each batch was practically constant. In the upper reaches of the curves there appears to be more of a tendency for the curves to flatten indicating somewhat less sensitivity to increased water. This is particularly true in the richer mixes and in the case of the ten, ½-in. drops.

Relative to the functioning of the flow-table when using the different operations there appears to be but one outstanding

difference. This difference is only a question of the magnitude of the flow (see Fig. 7) and exists between the five, and the ten, ½-in. drops. It would be expected that the greater number of drops would increase the diameter of the specimen but that the increase in flow should be constant regardless of wetness, mix, or the presence of an admixture, does not at first appear to be logical. The parallelisms existing in Figs. 3 and 4 are brought out in Fig. 7 where the flows for the five, and ten, ½-in. drops are plotted against the flows for the 15, ⅛-in. drops. The fact that the curve of flows for the ten drops is parallel to that for the five drops indicates that when the first five drops have been administered the characteristics of a mix as to wetness or differences in consistency have been brought out and that the additional drops have no part in differentiating between conditions of wetness.

It was pointed out that it was thought the more violent action of the ½-in. drops might bring out differences similar to those that had been observed between the flow obtained with ⅛-in. drops and the slump test. Quite the opposite is to be noted. The fact that the slope of the curves for the ½-in. drops is steeper than the 45 degree line which represents the flow for 15, ⅛-in. drops (see Fig. 7) indicates that the ½-in. drop is not so sensitive as the ⅛-in. drop, since slightly greater changes in flow were obtained for the latter for a given change in consistency than were obtained for the ½-in. drops. Because the several points plotted, representing the five mixes, arrange themselves in no definite order it would appear that rich mixes show no action on the flow-table different from lean mixes. It will be noted that the curves in the lower portion of Fig. 7 coincide with those at the top of the figure indicating that the above discussion applies also to concrete in which Celite is used as an admixture. The conclusions with reference to the functioning of the flow-table are strengthened by the fact that in both group of tests, with and without an admixture, the same condition obtains.

The distinct parallelism that exists between the different operations of the flow-table is to be seen from a comparison of the curves in Fig. 5 in which richness or relative quantity of cement used is plotted against the quantity of water required to give a certain flow. Here the quantity of water varied uniformly with the richness of the mix irrespective of the method used. This

straight line relation between richness of mix and gaging water for a given consistency parallels other tests of similar nature by Prof. D. A. Abrams.[2]

An examination of Fig. 8 will indicate that previous observations pertaining to the relation of slump to flow have been confirmed where the richness of the mix is increased and when the admixture is used. For a given flow the slump showed a distinct tendency to increase with increased quantity of cement and with the use of Celite. Within the range of plastic mixes this increase in slump is closely proportional to the cement content.

Results of the tests do not take into account such conditions as disruption of the sample and segregation of the ingredients. In the case of the lean mixes the lack of paste resulted in considerable crumbling of the sample regardless of the method of operating the flow-table. This, however, was more pronounced for the ½-in. drop than for the less violent ⅛-in. drop. Even in the case of the stiff rich mixes the severity of the larger drop was evidenced by a more pronounced breaking up of the sample.

Although no study was made to determine the time required for performing the flow test by any of the three different methods it seemed to require no more time to make the 15, ⅛-in. drops than was required for the five, ½-in. drops. This was due to the necessity for stopping rotation of the hand wheel just as each half-inch drop occurred, to prevent the vertical shaft of the table from striking on the cam. Even if there were a saving in time of operation the authors feel inclined to recommend the use of the 15, ⅛-in. drops because of its slightly greater sensitivity to change in wetness, because the action is less violent, and because of the smoother and apparently better functioning generally.

Though the strength study made was purely incidental to the consideration of the slump-flow relation, the data are of some interest because of the tendencies and conditions brought out. In Fig. 9 the strength obtained for the different mixes (average of ten tests each) are distinguished by different notation. The two higher strengths for each mix were obtained for concretes gaged to slumps of approximately 2 in. and 6 in. These strengths

[2] "Tests of Powdered Admixtures in Concrete"—Proceedings Am. Society Testing Materials Vol. XXIX, Part II, p. 643, 1929.

alone have been used in locating the curves both of which are of the form $S = \frac{A}{B^x}$.

It is to be noted that, for the wide range of mixes considered, when the concrete was gaged to reasonable wetness or to give moderate slumps the resulting strengths conformed quite closely to a water-cement ratio-strength relation. When the slump was increased to about 8 in., requiring a larger amount of mixing water, the strengths did not conform to this relation and indicate that, for a given mix, the strength is approximately inversely proportional to quantity of water used. This relation appears to vary with the richness of the mix and to be more critical the richer the mix. This brings out the marked advantage to be obtained by limiting the wetness to moderate slumps in practice when high strengths are required. Increasing the wetness to the extent indicated in the case of the three richest mixes caused a 50 per cent reduction in the strength based on the potential strength at 28 days when moderate slumps were used. When reasonable consistencies are used the resulting strength may be indicated by the water-cement ratio-strength relation over a wide range of mixes.

With reference to the concretes in which Celite was used, it will be noted that the strength for a given water-cement ratio was higher when the admixture was used. When gaging the concretes to the same consistency it was necessary to add a certain amount of water to compensate for the absorption of the admixture. When correction is made for this it will be noted that the strength of the Celite concrete is equal to that of the plain concrete. It is to be noted that similar indications with regard to changes in water-strength relation with change in mix were obtained for the concretes containing Celite as were secured for the plain mixes.

CONCLUSIONS

Strength data obtained incidental to this study indicate that within the region of reasonable wetness, about 2- to 6-in. slump, the strengths conform quite closely to a water-cement ratio-

strength relation of the form $S = \frac{A}{B^x}$. Beyond the 6-in. slump, however, a separate water-cement ratio-strength relation appears to exist which varies with the mix and which is more sensitive to change in water in the richer mixes.

Though the concretes containing Celite produced higher strengths for a given water-cement ratio, when brought to the same slump by increasing the water somewhat to compensate for the absorption of the admixture, the strengths were equal to those obtained for plain concrete.

Although the slump test is without question more convenient for use in the field than is the flow-table, it is uncertain in its functioning particularly within the range where a sensitive and accurate determination of the wetness is desirable. This appears to be due to the slump's ceasing to function in one manner and beginning to function in another as evidenced by the shape or form of the specimen after slumping.

The results indicate that the flow test is more sensitive than the slump test in differentiating between changes in the wetness of concrete with changes in the quantity of water used. The change in flow within reasonable limits is approximately proportional to change in water regardless of the mix. For a given wetness or flow the quantity of water required is proportional to the quantity of cement used except possibly in the case of very lean mixes.

Using drops of different magnitude and varying the number of drops indicated that the change in flow with change in water was approximately the same for all methods used. The 15, ⅛-in. drops was somewhat more sensitive than the five or the ten, ½-in. drops, giving slightly greater increments of flow for a given change in water content.

For a given wetness or consistency as measured by the flow the slump changed considerably and quite consistently with change in richness of mix, increasing as the quantity of cement increased and when Celite was used to improve the workability.

The data on the whole indicate that, where a consistent indication of the consistency or wetness of concrete is desired, the flow test is preferable to the slump test because of its more nearly

uniform functioning. Because of easier action and greater sensitiveness to change in wetness the 15, $\frac{1}{8}$-in. drops are to be preferred to either the five, or the ten, $\frac{1}{2}$-in. drops.

A Study of Slump and Flow of Concrete

BY INGE LYSE* AND W. R. JOHNSON†

INTRODUCTION

The first efforts to control the quantity of mixing water in concrete were directed toward the development of some method of measuring the consistency. The slump test was the first method that met with any general application. This was followed by the development of the flow-table. Both these tests, which have been standardized by the American Society for Testing Materials, have been quite widely used for the control of consistency in laboratory investigations and to some extent in the control of consistency in the field. The slump test has achieved a much wider use in the field than has the flow-table, largely because of the greater simplicity in the apparatus required.

One of the earliest uses of these methods for measuring consistency was in the study of integral admixtures. Neither method, however, met with general favor in this field nor have they been widely accepted as suitable measures of the elusive property called workability. The deficiencies of these methods have been further emphasized by the recent development of the water-cement ratio specification for concrete in which the requirement of "plastic and workable" mixes is substituted for a direct specification of both grading and proportions of aggregates.

In spite of the recognized deficiencies of these methods, there is still much to commend them, and pending the development of a more applicable form of measurement it seemed desirable to investigate somewhat more thoroughly than has been done heretofore, the possibilities of these methods and to obtain a direct comparison between the two. The tests reported in this paper were undertaken with this in view, as a part of a general investiga-

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tion of admixtures under way in the Research Laboratory of the Portland Cement Association. The tests were carried out in such a way that in addition to the direct comparison between slump and flow under the Standard A. S. T. M. procedure, various other comparisons were possible. In the flow test, readings were taken at 5, 10, 15 (standard) and 25 drops of the flow-table. Also the slump of the flow cone was measured just as in the slump test. This was done when the form was first removed and at the time of each successive reading of the flow. In addition, the specimen formed by the slump cone was treated in the same way, reading slumps and flow after the various number of drops. This gave an opportunity to observe the behavior of the concrete under the action of the flow-table for both types of specimens and the readings provide a numerical basis upon which the methods can be compared.

OUTLINE OF TEST

The cement used in this series of tests consisted of a mixture of equal parts of four brands of portland cement purchased in Chicago. The aggregate was calcareous sand and gravel from Elgin, Illinois. The sand was naturally graded 0-No. 4 and the gravel was No. 4-1½ graded as follows: 25 per cent No. 4-⅜ in., 50 per cent ⅜-¾ in., and 25 per cent ¾-1½ in. The concrete was mixed by hand in batches sufficiently large to fill one 6- by 12-in. cylinder. Two batches were made of identical mix. On the first batch, slump was measured in accordance with standard method. In this test the slump cone was supported on the 30-in. flow-table so that immediately following the measurement of the slump the table was lifted and dropped in exactly the manner used in the standard flow test, the height of drop being ½ in. Both the slump and the flow or the spread of the concrete cone were measured after dropping the concrete for 5, 10, 15 and 25 times. In this manner both the flattening and spread of the slump cone were obtained for different number of jiggings on the flow-table.

The next batch of identical concrete was placed in the standard flow cone and both flattening of the cone and the flow were taken for 0, 5, 10, 15 and 25 drops of the flow-table respectively. Each test was repeated on three different days and the averages used for this study.

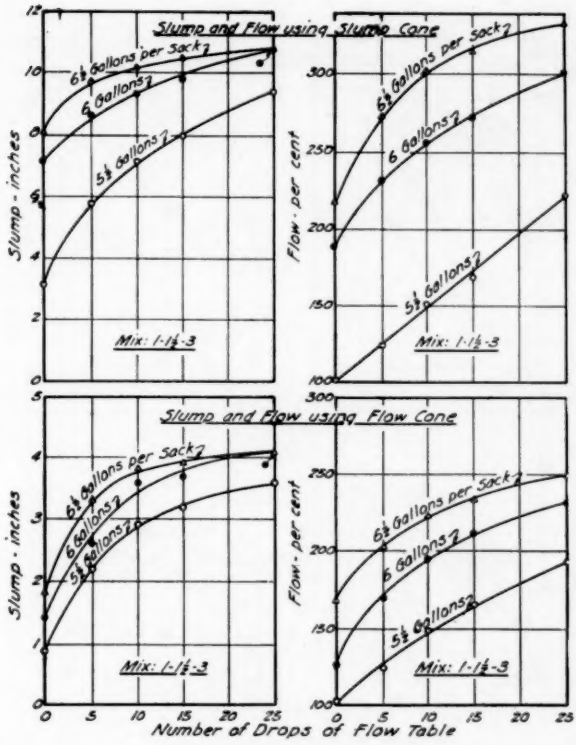

FIG. 1—EFFECT OF NUMBER OF ½-IN. DROPS OF FLOW-TABLE ON SLUMP AND FLOW OF CONCRETE

Five different concrete mixes were used: 1:1½:3, 1:2:3, 1:2:4, 1:2½:4 and 1:2½:5, by compacted volumes. Of these mixes the 1:1½:3, 1:2:4 and 1:2½:5 were repeated with the addition of 2, 5 and 10 per cent (by weight) of Celite, Pumicite and hydrated lime respectively, and with 2, 5, 10 and 20 per cent of portland cement as admixture. At least 3 different water-cement ratios were used for each mix, percentage and kind of admixture. The admixture was added to the dry materials before mixing.

TEST DATA

A complete set of test data is given in Tables 1 and 2. Table 1 gives the data obtained for slump and flow using the slump cone,

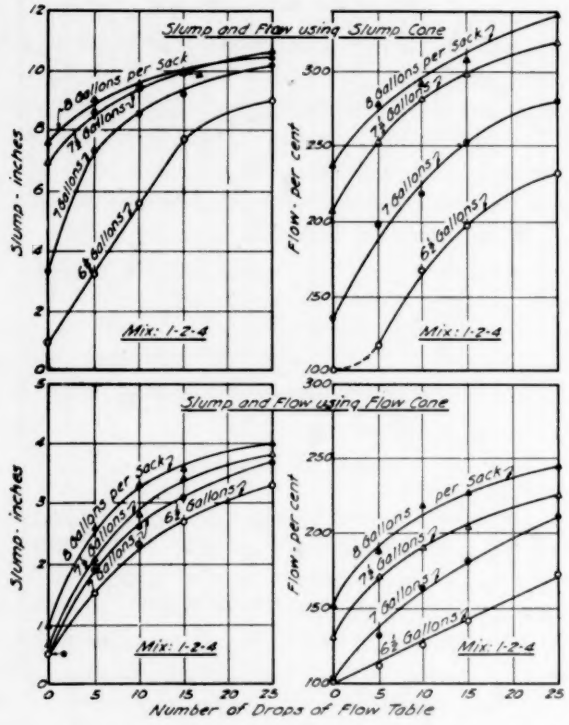

FIG. 2—EFFECT OF NUMBER OF ½-IN. DROPS OF FLOW-TABLE ON SLUMP AND FLOW OF CONCRETE

and Table 2 the corresponding values with the standard flow cone.

The effect of the number of ½-in. drops of the flow-table on slump and flow is shown in Figs. 1 to 3. The upper diagrams in these figures represent the values obtained with the slump cone while the lower diagrams show the corresponding values for the flow cone. Fig. 1 represents a concrete mix of 1:1½:3 by compacted volumes; Fig. 2, a 1:2:4 mix; and Fig. 3, a 1:2½:5 mix. The effect of an addition in water content on slump and flow is also seen from these figures.

Fig. 4 shows a comparison between flows obtained with the flow cone and with the slump cone for 5 drops of the flow-table.

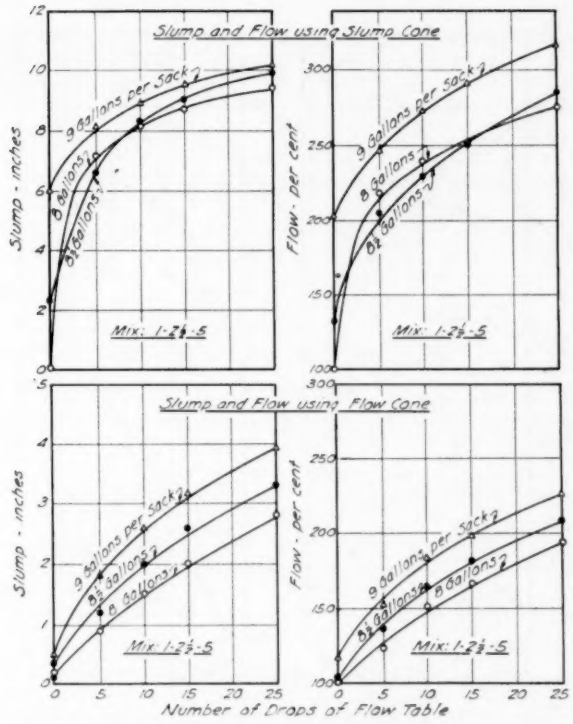

FIG. 3—EFFECT OF NUMBER OF ½-IN. DROPS OF FLOW-TABLE ON SLUMP AND FLOW OF CONCRETE

Figs. 5 to 8 give the relation between the standard methods of measuring slump and flow for plain concrete mixes and for concrete containing portland cement, Celite, Pumicite, and hydrated lime admixtures respectively, and Figs. 9 to 12 show a comparison between slump measured in the standard manner and flow measured with the slump cone after five ½-in. drops on the flow-table using concrete mixes containing the above admixtures. Fig. 12 and 13 give the effect of the admixture on both slump and flow of concrete for given water-cement ratios.

DISCUSSION OF DATA

Effect of Number of Drops of Flow-Table

Each ½-in. drop of the flow-table will shake the concrete to

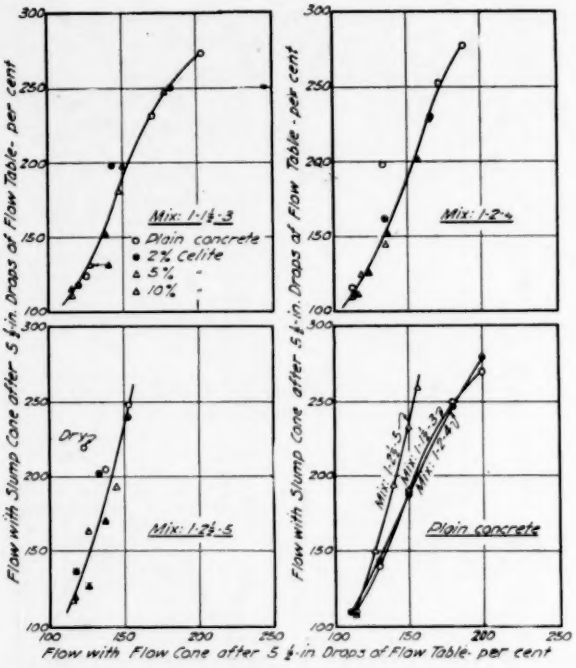

FIG. 4—RELATION BETWEEN FLOW OF CONCRETE USING FLOW CONE AND SLUMP CONE

such an extent that both the flattening and the spread of the cone will increase. Both slump and flow will therefore increase with the number of drops of the flow-table when using either cone as shown in Figs. 1 to 3. This increase will be rather uniform if the mix is fairly plastic and workable. With a harsh or very dry mix, a crumbling of the materials takes place, giving a sudden increase in both the slump and flow. This is more pronounced with the slump cone specimen. A typical example of the effect of crumbling on the slump and flow obtained with the slump cone is shown in Fig. 3 for 5 drops of the flow-table for a 1:2½:5 mix having 8 gal. of water per sack of cement. The crumbling of the cone during the shaking of the first few drops gave a large index of flow even though no actual flow of the concrete occurred. Slump

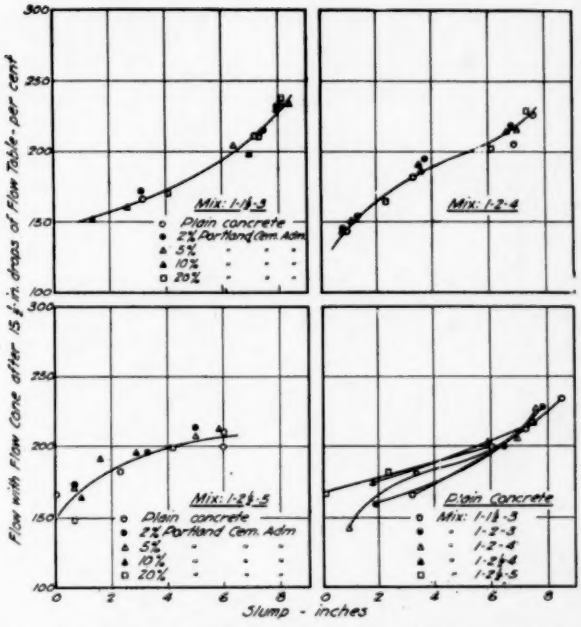

FIG. 5—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

or flow obtained on concrete which crumbles may be misleading when judging the consistency or placeability of concrete mixes.

In most cases the increase in slump and flow is greater for the first few drops of the flow table than for a larger number. Practically all the curves therefore have a steeper inclination for a small number than for a larger number of drops. This applies to both slump and flow with either cone. In the case, however, of the slump as measured on the slump cone, the curves also approach each other as the number of drops is increased. This means that the dry and stiff mixes show a larger increase in the slump with the increase in shaking of the concrete than do the wet and more plastic mixes. The sensitivity of the slump is therefore decreased with the number of drops. The resulting conclusion is that a slump test should be taken with no shaking

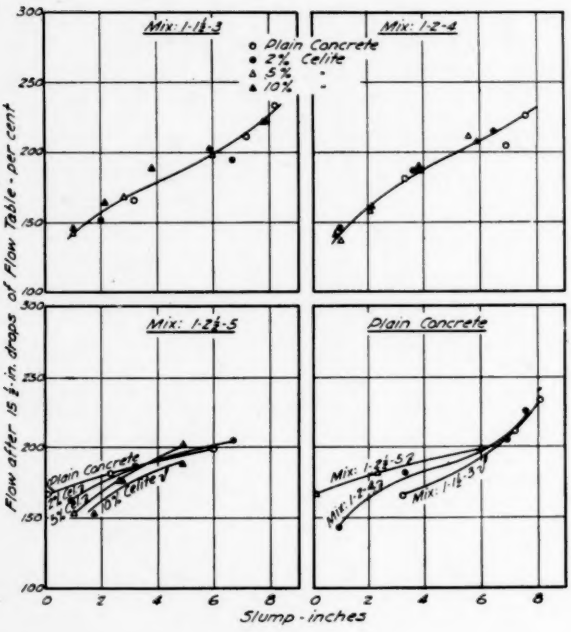

FIG. 6—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

of the flow table, which corresponds to the standard method of making the slump test.

The slumps measured with the flow cone, however, show a somewhat different behavior. The slumps on plastic mixes seem to increase with the increase in shaking just as fast and in some cases even faster than do the slumps for relatively stiff mixes. This difference in behavior is mostly due to the difference in shape of the concrete cone used, and indicates that for a flow cone the use of a large number of drops on the flow-table will not decrease the sensitivity of the slump materially.

The increase in number of drops of the flow-table seems to have little or no effect upon the sensitivity of the flow as measured either with the slump cone or flow cone. The curves remain

practically parallel to each other throughout the range used in this series so that no definite number of drops indicates any special advantage. It appears therefore of little importance at what number of drops the flow is taken; five drops seem to give just as sensitive a measure of the flowability of the concrete as the standardized 15 drops. Figs. 1 to 3 also show the effect of the water content on the consistency of concrete. In general the concretes of different water contents behaved similarly under increasing number of drops of the flow-table.

Slump Using Slump Cone and Flow Cone

The slump measured with the 12-in. slump cone of 8-in. base diameter is naturally much larger than the slump measured on the 5-in. flow cone of 10-in. base diameter. Figs. 1 to 3 show that the slump with the slump cone is usually several times as large as that with the flow cone for concrete of a given composition. The difference in slump for different water content is also correspondingly larger for the slump cone than for the flow cone. The slumps with the flow cone are actually so small that this method of test is practically useless for measuring plasticity of concrete. Just a glance at the curves in Figs. 1 to 3 shows the slump as measured on the standardized cone of the American Society for Testing Materials to be much more suitable than when measured on the flow cone.

Flow Using Slump Cone and Flow Cone

The flow with the slump cone is ordinarily much larger than with the flow cone for the same mix and number of drops. The difference in measured flow caused by changes in the water content is also considerably larger for the slump cone than for the flow cone. This is shown by Figs. 1 to 3 and leads to the conclusion that the slump cone provides a more sensitive test than the flow cone on the flow-table with ½-in. drops.

In Fig. 4 a direct comparison has been made between flows with the slump and flow cones using five ½-in. drops. The steep inclination of the curves indicates the much larger percentage of flow with the slump cone than with the flow cone. For the 1:1½:3 and 1:2:4 mixes which show an almost identical relation between flow with the two cones, a flow of 150 per cent on the flow cone corresponds to 190 per cent on the slump cone. This ratio re-

mains practically constant at all flows, indicating that the slump cone is a more sensitive measure of the changes in flowability of concrete than the flow cone. The addition of up to 10 per cent Celite by weight of cement does not seem to make any change in this relation. Neither of the two methods, therefore, shows any particular advantage over the other for measuring the smoothness of the mix produced by the addition of the Celite.

In the lower right hand corner of Fig. 4 each of the three plain concrete mixes is shown. It is noted that the 1:2½:5 mix has a considerably steeper inclination than the 1:1½:3 and 1:2:4 mixes. The relation between flow obtained with the flow cone and that obtained with the slump cone, therefore, varies somewhat with the richness of the mix. This affords a direct comparison between the suitability of each of these two methods as a measure of change in workability produced by changes in mix. For a given flow with the slump cone the leanest mix gives the lowest flow with the flow cone while for a given flow on the basis of the flow cone the richest mixes give the lowest flow with the slump cone. According to a general conception of workability of concrete a rich mix having the same flow on the standard flow test as a lean mix, will be the most workable of the two. Correspondingly, the flow cone will be selected as the one giving a better indication of real workability than the slump cone for use on the flow table. This fact partly offsets the advantage in the greater sensitivity of the slump cone found above.

Relation Between Standard Methods of Measuring Slump and Flow of Concrete

The relation between the two methods for measuring workability of concrete standardized by the American Society for Testing Materials is given in Figs. 5 to 9. Fig. 5 shows the relation for plain concrete mixes and concrete containing up to 20 per cent portland cement as an admixture. It is noted that the curve representing the relation between slump and flow changes somewhat with the richness of the mix. The addition of portland cement in the quantities used, however, is not large enough to make any material change in this relation. The two methods are therefore about equally suitable for measuring the effect of an

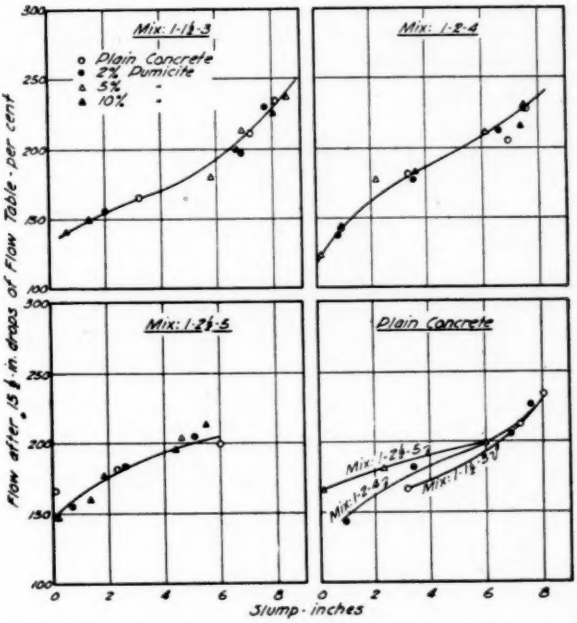

FIG. 7—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

addition of cement used as an admixture. For a mix as rich as 1:1½:3 the sensitivity of the flow as compared with the slump is not so large for dry consistencies as for wet consistencies. For leaner mixes, however, the relation is reversed and the sensitivity of the flow seems to be greatest for rather dry consistencies.

In the lower right hand corner of Fig. 5 the different mixes of plain concrete have been grouped together. This figure shows very clearly the variation in the relation between slump and flow caused by the richness of the concrete mix. It is generally considered that of two mixes having equal slump the richer mix is the more workable. A comparison between the 1:1½:3 mix and the 1:2½:5 mix shows that for the same slump the flow of the 1:1½:3 mix is considerably less than the flow of the 1:2½:5 mix. This is entirely contrary to a general conception of workability. If on

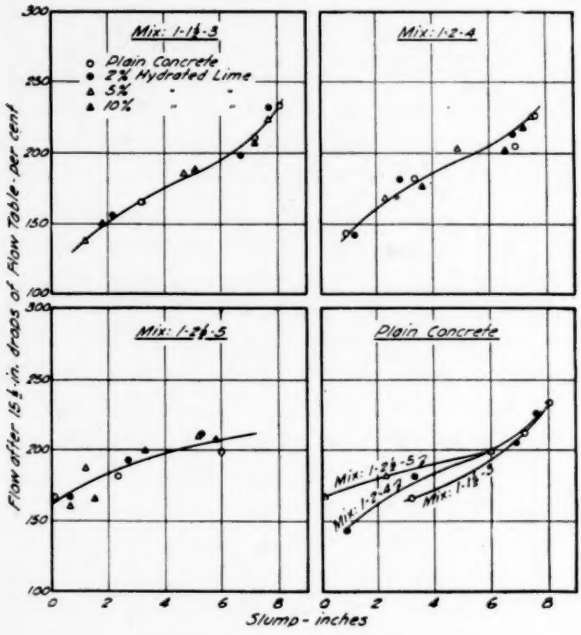

FIG. 8—RELATION BETWEEN STANDARD MEASURES OF SLUMP AND FLOW OF CONCRETE

the other hand a comparison is made on the basis of equal flow the richer mixes are seen to have the greatest slump. This strongly supports the conclusion that the slump is superior to the flow as an indication of the degree of workability of concrete mixes.

Fig. 6 shows the relation between the standard slump and flow when Celite is used as an admixture in amounts up to 10 per cent. The results indicate little or no change in the relation between slump and flow from the addition of Celite to 1:1½:3 and 1:2:4 mixes. The 1:2½:5 mix, however, was more affected by the Celite admixture; each increase in the amount of Celite gave a different relation between slump and flow. For a given flow the slump was increased with each addition of admixture. Thus the smoothing effect of the admixture changes the slump to a greater

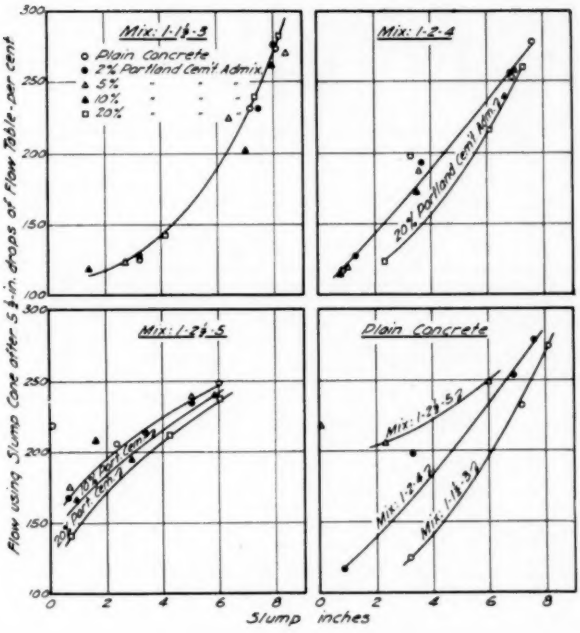

FIG. 9—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

extent than the flow so that the slump here also might be considered the more satisfactory measure of the workability of concrete.

Fig. 7 shows the relation between standard slump and flow for admixtures of Pumicite, and Fig. 8 for admixtures of hydrated lime. The effect of both these admixtures is seen to be very small for both the 1:1½:3 and 1:2:4 mixes, and even for the 1:2½:5mix the relation is practically unchanged. This indicates that either slump or flow is equally suitable for measuring the smoothing effect of these admixtures. This smoothing effect, therefore, appears to be less than that found by the Celite. This does not alter the conclusion drawn from other data obtained in

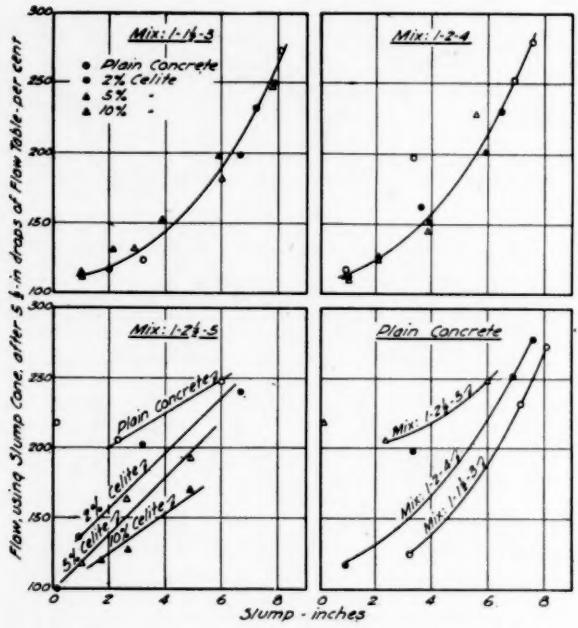

FIG. 10—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

this series of tests that the slump gives a more accurate indication of the real workability of a concrete mix than the flow.

Relation Between the Standard Slump Test and the Flow Using the Slump Cone After 5 Drops with Flow-Table

In the above discussion of the relation between flow measured with the flow and slump cones it was found that the slump cone gave the more sensitive measure, while the flow cone gave a somewhat better indication of real workability based on the richness of the mix. In view of the greater sensitivity shown by the flow when measured on the slump cone specimen, it was of interest to compare the results on this basis with those obtained by the standard slump test. This comparison is shown in Figs. 9 to 12 where the flow is based on 5 drops, using the slump cone specimen. These figures can be compared directly with Figs. 5 to 8 in which

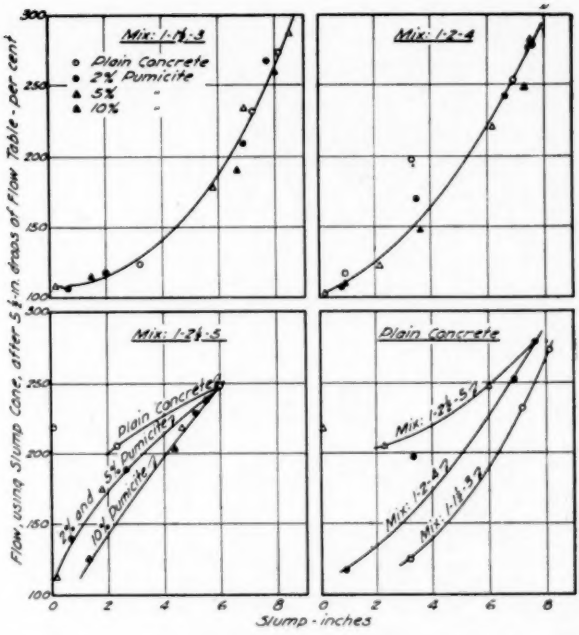

FIG. 11—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

the relation is based on the standard flow. In this comparison it will be seen that the curve in the pairs of figures: 5 and 9; 6 and 10; 7 and 11; and 8 and 12 are very similar in their general characteristics. They show two significant characteristics: First; for the rich mixes the flow is more sensitive than the slump in the case of the wet consistencies, but less sensitive in the case of the dry consistencies. Second; for the intermediate and lean mixes, the relationship is just reversed, the flow being less sensitive than the slump for the wetter mixes and more sensitive for the stiff mixes.

A further comparison of these pairs of figures brings out in an indirect way that the standard flow is a better measure of real workability than the flow using 5 drops with the slump cone specimen. This refers to improvement in workability, both by

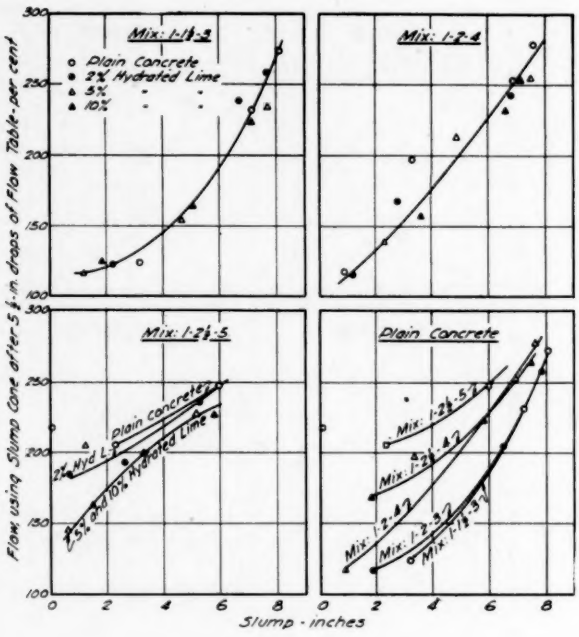

FIG. 12—RELATION BETWEEN SLUMP AND FLOW AS MEASURED WITH SLUMP CONE

increasing the cement content and by the use of some admixture which has a smoothing effect. In the discussion of Figs. 5 and 6, it was shown that for a given flow there was an increase in slump with an increase in richness for the plain concrete and an increase in slump with increasing percentages of Celite. This, it was pointed out, indicated the superiority of the slump as a measure of workability. In a study of Figs. 9 and 10 the same indication is seen in a much more marked degree, the slump being very plainly a better measure than the modified flow test of the increase in workability produced either by increasing the cement content or addition of Celite. Thus indirectly it is seen that in the flow test the standard flow cone gives a better indication of workability than the slump cone.

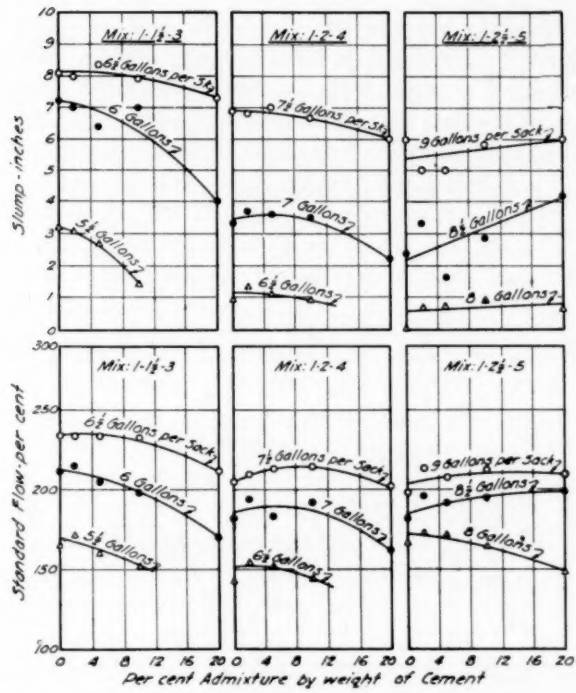

FIG. 13—EFFECT OF PORTLAND CEMENT AS AN ADMIXTURE ON SLUMP AND FLOW OF CONCRETE

In comparing Figs. 11 and 12 with Figs. 7 and 8, it is seen that in using the modified flow test, the relation between slump and flow is affected by additions of both Pumicite and hydrated lime, and also that the spread of the curves, representing the concretes without admixture, is very much greater. Both of these facts indicate the superiority of the slump test over the modified flow in bringing out the greater workability produced by enriching the mix or addition of admixture.

Effect of the Amount of Admixture on the Slump and Flow of Concrete

In the previous discussion it has been pointed out that the slump is a somewhat better measure than the flow of the improve-

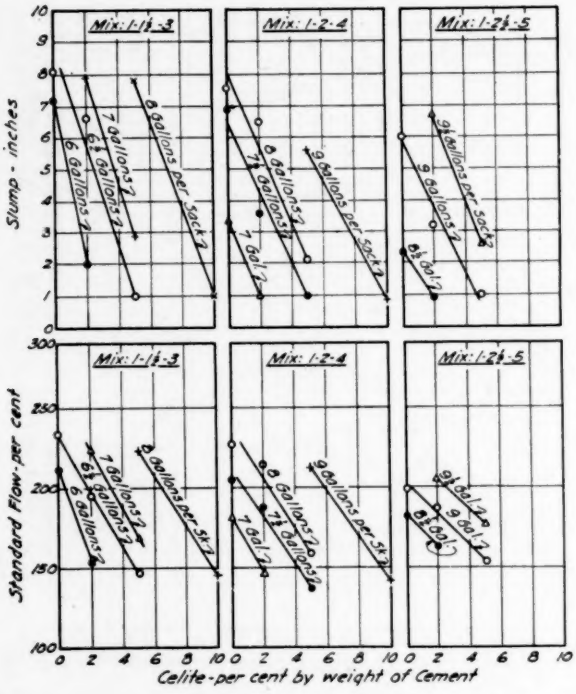

FIG. 14—EFFECT OF CELITE ON SLUMP AND FLOW OF CONCRETE

ment in smoothness resulting from the addition of admixtures to concrete. The actual effect of the amount of admixture on either the slump or flow, however, was not brought out. In Figs. 13 and 14 data are presented to show the effect of varying amounts of portland cement as an admixture and Celite on both the standard slump and standard flow for the several mixtures used in these tests. The data for the other admixtures are not shown as the curves for these lie between those for portland cement and Celite.

In Fig. 13 for portland cement as an admixture the upper diagrams contain the relations for the slump, and the lower ones those for the flow. It is noted that for a given water-cement ratio

(based on the original amount of cement) the slump is slightly decreased for the 1:1½:3 mix as the amount of cement is increased. A similar relation is seen to exist for the 1:2:4 mix but the decrease in slump is much less. The decrease in the slump for the 1:2:4 mix is negligible for additions of cement up to 10 per cent and only a slight decrease is noted for a 20 per cent addition. The 1:2½:5 mix, however, shows a different relation as the slump is somewhat increased with the increase in amount of cement. This indicates that the improvement on smoothness is not quite as great as the drying effect on rich mixes but greater than the drying effect on the lean mixes.

The results from the flow tests show a relation between the flow and amount of admixture for concrete of given water-cement ratio generally similar to that between the slump and amount of admixtures. The addition of cement decreases the flow of a rich concrete, but affects the leaner mixes to a lesser degree. For the 1:2½:5 mix the increase in flow with the addition of cement is slightly less than the increase in slump, indicating the previous finding that the slump is a more sensitive test than the flow for that smoother working which improves the workability of concrete.

In Fig. 14 the effect of Celite on a concrete mix of given water-cement ratio is shown. The slump as well as the flow is seen to decrease very sharply with the addition of the Celite. The upper diagram shows that for a 1:1½:3 mix of 6-gal. water ratio a slump of 7 in. in the plain concrete is reduced to 2 in. by the addition of 2 per cent Celite and an 8-in. slump in a 6½-gal. water ratio is reduced to 1 in. by adding 5 per cent Celite to the mix. To maintain a slump of about 7 in. in a 1:1½:3 mix the use of 2 per cent Celite requires the addition of ½ gal. of water per sack of cement and 6 per cent Celite requires an addition of 2 gal. over that required by concrete without admixture. The 1:2:4 and 1:2½:5 mixes show similar drops in slump with increase in percentage of Celite. The decrease in slump seems to be about proportional to the increase in percentage of admixture.

The relation between flow and per cent of Celite is shown in the lower diagrams of Fig. 14. Relations almost identical with those for the slump are found for the flow. The drying effect of

the admixture therefore affects both measures of consistency in a similar manner, and neither measure appears to be the better in comparing changes in consistency due to changes in water content. Fig. 6, however, showed the slump to be the more suitable measure of changes in workability due to admixtures, especially for the lean mixes. Fig. 5 (lower right hand diagram) also showed the slump test to be the better measure of increased workability due to increasing cement content.

The large difference between the curves in Figs. 13 and 14 is due to the kind of admixture used. While the addition of small percentages of cement did not change the consistency of a given mix to any important extent (water content unchanged) a very noticeable stiffening of the mix did result from the addition of corresponding percentages of Celite. Pumicite and hydrated lime did not show quite as great a drying effect as the Celite so that the relation between the slump and flow and the amount of these admixtures lies somewhere between the extremes represented in Figs. 13 and 14 for cement and Celite respectively.

SUMMARY

All data included in this series of tests show that the slump gives a more suitable indication than the flow of the increased workability of concrete produced either by enriching the mix or additions of admixtures. The results also indicate that the slump should be taken with no shaking of the concrete. The flow, however, remains about constant for the different number of drops on the flow-table so that any number is about equally suitable for the test. The use of five, ½-in. drops of the flow-table would seem to be as satisfactory as the use of 15, and would noticeably shorten the test.

The slump with the flow cone is not a sensitive test and is not suited to measuring consistency or workability.

The flow test is much more sensitive with the slump cone than with the standard flow cone for measuring changes in consistency due to changes in water content, also in showing the effect of increasing number of drops of the flow table. The use of the standard flow cone, however, seems to give a more accurate indication of increased smoothness resulting from richer mixes or the use of admixtures. This, therefore, partly offsets the ad-

vantage of the higher sensitivity obtained with the slump cone when used for measuring flow.

The relation between the flow and slump measured in accordance with the methods standardized by the American Society for Testing Materials changes with the richness of the concrete mix. For concrete of a given flow the slump increased as the richness increased, indicating that the slump is the better of the two methods for measuring improvement in workability obtained through increased cement content. The addition of admixtures also affects the slump to a greater extent than the flow, especially for lean mixes, which indicates further the superiority of the slump test over the flow test.

Modifying the flow test by using the slump cone instead of the standard flow cone increased the sensitivity of the test to changes in water content, but not to improvement in workability obtained through richer mixes or the use of admixtures.

While these tests have shown an increase in smoothness of concrete due to the use of admixtures they have also shown the stiffening of the mix which results from their use when no change is made in the water content.

(*See Tables 1 and 2 on following pages*).

Readers are referred to the JOURNAL *for May, 1931, for discussion of the two preceding papers, which may develop. Such discussion should reach the Secretary by April 1, 1931.*

A STUDY OF SLUMP AND FLOW OF CONCRETE

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE

Mix by volume: sufficient material used to make one 6- by 12-in. cylinder.

Cement: a mixture of equal parts of 4 brands of portland cement purchased in Chicago.

Aggregate (room dry when used):

Fine: sand from Elgin, Ill., graded from 0 - No. 4 (F. M. = 3.00).

Coarse: pebbles from Elgin, Ill., graded from No. 4 - 1½ in. (F. M. = 7.00).

Flow and slump tests made immediately after mixing concrete. A 4 - by 8- by 12-in. slump cone was filled in 3 layers on a 30-in. flow table, each layer being rodded 25 times. Immediately after filling the cone was removed, both slump and flow recorded and the table was then given five ½-in. drops in about 3 seconds and the slump and flow again recorded. This operation was repeated on the same sample using 10, 15 and 25 ½-in. drops.

Each value is the average of 3 tests made on different days.

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In. Drop													
1:1½:3	5½	——	—	3.2	5.8	7.2	8.0	9.4	101	124	151	169	222
	6			7.2	8.6	9.3	9.8	10.8	189	231	257	272	301
	6½			8.1	9.7	10.2	10.5	10.8	218	273	302	316	335
1:2:3	6	——	—	1.9	4.6	6.2	7.4	8.9	100	117	139	163	200
	6½			6.5	8.3	9.4	10.1	10.8	150	204	238	263	295
	7			7.8	9.4	10.0	10.5	11.0	208	259	288	306	331
1:2:4	6½	——	—	0.9	3.2	5.6	7.7	9.0	100	117	167	197	232
	7			3.3	7.3	8.5	9.2	10.2	135	198	219	252	280
	7½			6.9	8.6	9.4	9.9	10.7	207	253	281	298	320
	8			7.6	9.0	9.5	9.9	10.5	238	278	292	309	338
1:2½:4	7½	——	—	1.8	6.0	7.9	8.7	9.8	100	169	205	230	263
	8			5.8	8.2	9.5	10.3	10.8	175	222	254	274	309
	8½			7.5	9.1	9.6	10.0	10.7	220	264	292	308	329
1:2½:5	8	——	—	0.1	7.2	8.2	8.7	9.4	100	219	240	252	275
	8½			2.3	6.6	8.3	9.0	9.9	132	205	230	251	285
	9			6.0	8.1	8.9	9.5	10.2	203	248	273	291	318
1:1½:3	6	Celite	2	2.0	4.8	6.6	7.6	9.0	101	118	143	158	197
	6½			6.7	8.5	9.5	10.1	10.6	141	199	231	259	292
	7			7.9	9.6	10.3	10.6	10.9	188	250	283	303	328
1:1½:3	6½	Celite	5	1.0	3.5	5.1	6.4	8.1	100	111	127	136	169
	7			2.8	6.1	7.9	8.8	9.9	101	132	156	183	227
	7½			6.0	8.6	9.9	10.3	10.8	123	181	220	254	295
	8			7.8	9.4	10.3	10.5	10.8	183	248	289	303	331
1:1½:3	8	Celite	10	1.0	3.5	5.1	6.4	8.3	100	116	131	149	177
	8½			2.1	5.7	7.7	8.8	10.0	100	132	159	187	203
	9			3.8	7.7	9.2	10.1	10.8	105	153	192	222	268
	9½			5.9	8.9	10.1	10.4	10.7	129	199	221	266	269

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In Drop													
1:2:4	7	Celite	2	1.0	3.6	6.4	7.8	9.3	100	111	164	187	221
	7½			3.6	7.1	8.4	9.3	10.4	101	161	198	229	268
	8			6.5	8.4	9.6	10.2	10.5	180	230	264	282	313
1:2:4	7½	Celite	5	1.0	3.4	5.3	6.7	8.1	100	110	132	154	182
	8			2.1	5.3	6.8	7.8	9.2	100	125	153	178	217
	8½			3.8	7.0	8.5	9.3	10.2	102	145	186	220	265
	9			5.6	8.6	9.9	10.0	10.5	172	229	265	283	312
1:2:4	9	Celite	10	0.8	3.0	4.7	5.8	7.9	100	112	128	141	172
	9½			2.1	5.6	7.4	8.7	9.9	100	127	158	183	232
	10			3.9	7.5	9.2	9.9	10.6	102	152	193	224	270
	10½			5.9	8.6	9.8	10.5	10.9	134	201	243	272	305
1:2½:5	8½	Celite	2	0.9	5.3	7.3	8.4	9.5	100	137	177	208	244
	9			3.2	7.0	8.6	9.4	10.1	125	202	225	250	285
	9½			6.7	8.6	9.5	10.0	10.8	185	240	273	287	315
1:2½:5	9	Celite	5	1.0	4.0	6.2	8.1	9.7	100	118	157	186	221
	9½			2.6	6.4	8.4	9.4	10.1	100	163	206	232	272
	10			4.9	7.7	8.9	9.6	10.2	131	194	235	259	294
1:2½:5	10½	Celite	10	1.7	4.8	6.7	7.9	9.3	100	120	146	170	216
	11			2.6	6.1	7.9	9.2	10.3	100	127	168	199	248
	11½			4.9	8.0	9.4	10.1	11.0	102	170	218	253	292
1:1½:3	5½	Pumicite	2	2.0	4.7	6.5	7.5	8.9	100	118	139	164	200
	6			6.9	8.6	9.4	10.1	10.7	150	210	241	263	296
	6½			7.7	9.4	10.1	10.6	10.9	221	267	294	311	332
1:1½:3	5½	Pumicite	5	0.6	3.0	4.6	5.9	7.9	100	107	120	130	170
	6			5.8	8.1	9.2	9.9	10.5	129	179	214	243	278
	6½			6.9	8.8	9.6	10.4	10.7	186	234	263	283	309
	7			8.5	10.0	10.5	10.7	11.1	247	287	312	325	346
1:1½:3	6	Pumicite	10	1.4	4.0	6.0	7.0	8.4	100	115	131	150	181
	6½			6.7	8.6	9.8	10.3	10.7	137	190	229	255	293
	7			8.0	9.4	10.1	10.6	10.9	207	259	289	306	331
1:2:4	6½	Pumicite	2	0.8	2.6	5.3	7.0	8.6	100	109	133	152	187
	7			3.5	5.4	7.8	8.8	9.8	114	170	205	227	261
	7½			6.6	8.5	9.5	10.2	10.7	188	242	271	293	319
	8			7.5	9.1	9.7	10.3	10.8	235	277	296	315	328
1:2:4	6½	Pumicite	5	0.2	1.8	3.4	5.8	7.7	100	104	120	148	186
	7			2.2	5.2	7.0	8.0	9.5	100	121	172	199	243
	7½			6.2	7.9	8.9	9.6	10.4	173	220	252	273	303
	8			7.5	8.9	9.6	10.3	10.6	239	281	305	323	339
1:2:4	7	Pumicite	10	0.9	3.3	5.5	6.8	8.5	100	110	133	155	194
	7½			3.6	6.2	8.0	9.1	10.2	105	147	187	222	263
	8			7.3	9.3	10.0	10.6	11.0	198	249	277	299	326
1:2½:5	8	Pumicite	2	0.7	3.4	6.2	7.6	9.0	100	140	198	215	244
	8½			2.6	5.8	7.4	8.3	9.6	136	189	215	236	266
	9			5.1	7.7	8.8	9.4	10.3	191	229	256	279	306
1:2½:5	8	Pumicite	5	0.2	4.3	6.4	7.2	8.3	100	111	162	181	216
	8½			1.8	6.1	7.7	8.6	9.5	113	175	212	233	268
	9			4.6	7.0	8.3	9.0	9.9	176	219	246	269	298
1:2½:5	8½	Pumicite	10	1.3	4.7	6.7	7.9	8.9	100	126	183	209	243
	9			4.4	6.9	8.3	9.1	9.9	152	203	230	255	289
	9½			5.5	7.7	8.6	9.4	10.2	194	239	270	282	316

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In. Drop													
1:1½:3	5½	Portland	2	3.2	6.1	7.6	8.5	9.7	101	128	153	184	228
	6	Cement		7.5	9.0	9.5	10.1	10.8	184	231	262	283	312
	6½			8.0	9.4	10.3	10.5	10.9	233	276	301	318	341
1:1½:3	5½	Portland	5	2.7	5.5	7.1	8.2	9.3	101	122	146	167	207
	6	Cement		6.4	7.7	9.3	10.0	10.5	176	225	254	276	303
	6½			8.4	9.6	10.2	10.6	10.9	220	270	296	314	339
1:1½:3	5½	Portland	10	1.4	4.4	6.3	7.3	8.8	101	119	134	151	184
	6	Cement		7.0	8.6	9.4	9.8	10.6	153	203	236	259	292
	6½			7.9	9.5	10.4	10.4	11.0	215	261	289	307	328
1:1½:3	6	Portland	20	4.1	7.1	8.5	9.3	10.4	107	142	170	194	236
	6½	Cement		7.3	8.9	9.6	10.2	10.5	185	240	269	289	314
	7			8.2	9.7	10.4	10.6	11.0	233	282	308	325	344
1:2:4	6½	Portland	2	1.3	4.1	5.9	7.9	9.3	100	128	172	204	237
	7	Cement		3.7	7.3	8.6	9.3	10.1	130	194	225	255	285
	7½			6.8	8.4	9.4	10.0	10.6	203	255	281	300	325
1:2:4	6½	Portland	5	1.1	3.8	6.5	7.6	8.9	100	119	162	191	226
	7	Cement		3.6	6.8	8.2	8.9	9.7	125	188	219	241	277
	7½			7.0	8.7	9.6	10.3	10.7	209	258	284	302	328
1:2:4	6½	Portland	10	0.8	3.3	5.7	7.5	9.1	100	116	153	181	215
	7	Cement		3.5	6.8	8.6	9.4	10.6	119	172	207	231	272
	7½			6.7	8.8	9.7	10.2	10.9	186	240	270	290	316
1:2:4	7	Portland	20	2.3	5.3	7.0	8.1	9.4	100	123	156	182	223
	7½	Cement		6.1	8.3	9.4	9.9	10.7	163	217	247	270	300
	8			7.3	9.1	10.0	10.3	10.8	217	260	286	300	324
1:2½:5	8	Portland	2	0.7	4.5	7.2	8.2	9.3	103	168	211	232	258
	8½	Cement		3.3	6.8	8.1	9.1	10.2	173	213	235	193	295
	9			5.0	7.4	8.6	9.2	9.9	194	234	261	281	311
1:2½:5	8	Portland	5	0.7	5.3	7.9	8.9	9.8	100	174	203	224	260
	8½	Cement		1.6	6.4	8.3	9.0	9.9	148	209	241	265	292
	9			5.0	8.3	8.9	9.6	10.5	192	240	270	290	317
1:2½:5	8	Portland	10	0.9	4.8	6.9	8.0	9.1	100	167	207	221	256
	8½	Cement		2.8	6.2	7.9	8.7	9.9	133	195	221	250	280
	9			5.8	7.9	8.9	9.5	10.3	193	240	268	288	314
1:2½:5	8	Portland	20	0.7	4.7	7.3	8.3	9.6	100	140	189	211	241
	8½	Cement		4.2	7.1	8.5	9.2	10.1	152	211	241	264	291
	9			6.0	7.9	9.0	9.7	10.5	190	238	263	281	311
1:1½:3	5½	Hydrated	2	2.2	5.0	6.9	8.0	9.3	100	123	144	167	210
	6	Lime		6.7	8.6	9.6	10.1	10.7	182	238	272	287	315
	6½			7.7	9.2	9.9	10.4	10.9	195	256	287	309	334
1:1½:3	5½	Hydrated	5	1.2	3.6	5.3	6.5	8.1	100	117	130	148	181
	6	Lime		4.7	7.6	8.9	9.7	10.3	108	154	189	222	266
	6½			7.7	9.3	10.2	10.4	10.8	175	234	266	290	318
1:1½:3	6	Hydrated	10	1.8	4.7	6.7	7.7	9.2	100	125	142	158	197
	6½	Lime		5.1	7.9	9.0	9.6	10.4	117	165	198	228	268
	7			7.2	8.9	9.7	10.1	10.7	162	223	265	280	308
1:2:4	6½	Hydrated	2	1.2	3.9	6.2	7.7	9.4	100	115	163	190	228
	7	Lime		2.8	6.6	8.3	9.2	10.3	100	169	207	236	276
	7½			6.8	8.7	9.6	10.2	10.8	189	243	272	293	317
1:2:4	7	Hydrated	5	2.3	5.7	7.6	8.7	10.0	100	139	181	207	249
	7½	Lime		4.9	7.9	8.9	9.6	10.6	139	214	246	268	296
	8			7.5	9.2	9.9	10.5	10.9	191	254	288	305	331

TABLE 1—DATA OF SLUMP AND FLOW TESTS USING SLUMP CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Slump In. (Drops)					Ratio of Final to Original Diameter of Slump Specimen Times 100 (Drops)				
		Type	%	0	5	10	15	25	0	5	10	15	25
Slump Cone—½-In. Drop													
1:2:4	7½	Hydrated	10	3.6	7.4	8.7	9.5	10.2	100	158	198	229	267
	8	Lime		6.6	8.4	9.3	10.0	10.6	175	231	263	283	308
	8½			7.2	9.1	9.8	10.3	10.8	198	253	281	298	324
1:2½:5	8	Hydrated	2	0.7	5.4	7.4	8.7	9.8	100	185	209	226	259
	8½	Lime		2.7	6.7	8.4	9.4	10.5	118	192	228	252	293
	9			5.3	7.6	8.7	9.5	10.4	191	238	265	286	320
1:2½:5	8	Hydrated	5	0.6	5.2	7.4	8.8	9.8	100	146	185	206	237
	8½	Lime		1.2	6.9	8.2	9.1	10.2	102	205	231	255	286
	9			5.2	7.9	8.9	9.6	10.5	177	228	257	278	311
1:2½:5	8½	Hydrated	10	1.5	5.5	8.3	9.1	10.1	100	164	203	227	264
	9	Lime		3.3	7.0	8.5	9.3	10.2	109	200	234	256	290
	9½			5.8	8.3	9.3	10.0	10.7	175	229	261	284	315

A STUDY OF SLUMP AND FLOW OF CONCRETE

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE

Mix by volume: sufficient material used to make one 6-by- 12-in. cylinder.

Cement: a mixture of equal parts of 4 brands of portland cement purchased in Chicago.

Aggregate (room dry when used):

Fine: sand from Elgin, Ill., graded from 0 - No. 4 (F. M. = 3.00).

Coarse: pebbles from Elgin, Ill., graded from No. 4 - 1½ in. (F. M. = 7.00).

Flow and slump tests made immediately after mixing concrete. A 6¾- by- 10- by- 5-in. flow cone was filled in 2 layers on a 30-in. flow-table, each layer being rodded 25 times. Immediately after filling the cone was removed both flow and slump recorded and the table was then given five ½-in. drops in about 3 seconds and the slump and flow again recorded. This operation was repeated on the same sample using 10, 15 and 25 ½-in. drops.

Each value is the average of 3 tests made on different days.

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In. Drop													
1:1½:3	5½	—	0	0.9	2.2	2.9	3.2	3.6	102	125	149	166	193
	6			1.4	2.6	3.6	3.7	4.1	128	170	195	211	232
	6½			1.8	3.3	3.8	3.9	4.1	169	203	223	234	250
1:2:3	6	—	0	0.6	2.1	2.9	3.2	3.7	101	121	140	160	189
	6½			1.0	2.6	3.2	3.8	4.0	104	143	173	200	225
	7			1.4	3.1	3.7	3.9	4.1	148	189	218	229	248
1:2:4	6½	—	0	0.5	1.5	2.3	2.7	3.3	101	112	128	143	174
	7			0.5	1.9	2.6	3.1	3.7	102	133	164	181	211
	7½			0.7	2.1	2.8	3.4	3.8	132	171	190	205	226
	8			1.0	2.6	3.3	3.6	4.0	151	189	219	227	245
1:2½:4	7½	—	0	0.5	1.9	2.6	3.0	3.6	100	125	152	174	204
	8			0.9	2.2	3.0	3.5	3.9	107	154	182	203	227
	8½			0.9	2.6	3.4	3.8	3.9	130	176	201	216	238
1:2½:5	8	—	0	0.2	0.9	1.5	2.0	2.8	100	124	151	167	194
	8½			0.3	1.2	2.0	2.6	3.3	103	137	165	181	209
	9			0.5	1.8	2.6	3.2	3.9	118	153	184	199	227
1:1½:3	6	Celite	2	0.5	1.8	2.5	3.0	3.5	100	120	136	152	179
	6½			1.0	2.5	3.2	3.6	3.9	102	143	173	195	218
	7			1.5	3.1	3.7	3.9	4.0	133	181	207	223	243
1:1½:3	6½	Celite	5	0.3	1.6	2.2	2.8	3.3	100	116	129	143	164
	7			0.7	2.1	2.9	3.3	3.8	101	127	149	169	200
	7½			1.0	2.7	3.4	3.7	4.0	102	146	178	197	224
	8			1.4	3.0	3.5	3.9	4.1	126	178	204	222	244
1:1½:3	8	Celite	10	0.5	1.8	2.3	2.9	3.4	101	115	132	146	169
	8½			0.6	2.2	3.0	3.4	3.8	101	127	147	165	195
	9			0.9	2.6	3.3	3.6	4.0	103	138	168	189	217
	9½			0.9	2.9	3.5	3.9	4.0	104	150	183	204	230

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In Drop													
1:2:4	7	Celite	2	0.3	1.5	2.2	2.7	3.2	100	114	130	146	174
	7½			0.6	2.2	3.0	3.5	4.0	101	135	168	188	218
	8			1.0	2.8	3.5	3.7	4.0	113	166	196	215	236
1:2:4	7½	Celite	5	0.3	1.3	2.0	2.5	3.3	100	113	126	137	160
	8			0.6	1.8	2.8	3.0	3.6	100	119	140	159	191
	8½			0.9	2.4	3.2	3.5	3.9	102	136	168	190	217
	9			0.9	2.6	3.4	3.6	4.0	112	165	196	212	233
1:2:4	9	Celite	10	0.3	1.7	2.4	2.8	3.4	100	115	129	141	164
	9½			0.5	2.0	2.8	3.2	3.7	100	124	142	162	192
	10			0.9	2.5	3.3	3.7	3.9	102	137	164	187	218
	10½			0.9	2.8	3.5	3.9	4.1	105	158	189	209	235
1:2½:5	8½	Celite	2	0.4	1.5	2.4	2.9	3.6	100	118	139	162	195
	9			0.6	2.1	3.0	3.4	3.9	100	132	164	187	216
	9½			0.6	2.1	3.0	3.6	3.9	110	153	187	205	230
1:2½:5	9	Celite	5	0.4	1.6	2.2	2.9	3.6	100	116	133	153	185
	9½			0.6	2.0	3.3	3.3	3.9	100	127	155	177	209
	10			0.9	2.3	3.2	3.5	4.0	101	145	179	203	225
1:2½:5	10½	Celite	10	0.6	1.9	2.5	3.0	3.5	100	118	136	153	185
	11			0.6	2.0	2.9	3.4	3.9	101	127	153	177	209
	11½			0.8	2.6	3.3	3.6	3.9	101	138	170	189	221
1:1½:3	5½	Pumicite	2	0.5	1.9	2.7	3.1	3.6	100	120	138	156	187
	6			1.1	2.5	3.2	3.6	3.9	107	148	176	196	223
	6½			1.6	3.1	3.7	3.9	4.3	157	195	218	230	247
1:1½:3	5½	Pumicite	5	0.5	1.6	2.3	2.9	3.4	100	114	127	141	165
	6			0.9	2.2	3.3	3.4	4.0	102	131	156	180	211
	6½			1.6	2.7	3.2	3.7	4.1	135	168	190	214	236
	7			2.0	3.2	3.8	4.0	4.1	177	207	224	237	255
1:1½:3	6	Pumicite	10	0.4	1.7	2.5	3.0	3.6	101	118	134	150	178
	6½			1.2	2.7	3.3	3.7	4.0	110	151	178	200	224
	7			1.5	3.1	3.7	3.9	4.1	148	187	213	225	245
1:2:4	6½	Pumicite	2	0.4	1.5	2.0	2.5	3.0	100	111	125	137	160
	7			0.5	2.0	2.8	3.2	3.9	100	129	155	177	207
	7½			1.0	2.5	3.3	3.6	4.0	128	164	198	212	234
	8			1.7	3.1	3.7	3.9	4.2	158	194	216	229	249
1:2:4	6½	Pumicite	5	0.2	1.0	1.4	1.8	2.5	100	106	116	124	143
	7			0.5	2.0	2.9	3.4	3.9	100	123	153	178	209
	7½			0.9	2.6	3.3	3.7	4.1	122	162	191	212	234
	8			1.3	2.9	3.6	3.9	4.1	152	190	213	230	247
1:2:4	7	Pumicite	10	0.5	1.5	2.3	2.8	3.3	101	115	131	144	178
	7½			0.7	2.1	3.0	3.4	3.8	101	129	157	183	215
	8			1.1	2.8	3.5	3.9	4.2	128	167	198	216	239
1:2½:5	8	Pumicite	2	0.4	1.3	2.1	2.6	3.3	100	113	131	155	190
	8½			0.5	1.8	2.4	2.9	3.7	104	129	163	184	215
	9			0.7	2.2	2.8	3.4	3.9	118	155	184	206	231
1:2½:5	8	Pumicite	5	0.4	1.5	2.2	2.6	3.2	100	112	129	148	186
	8½			0.4	1.7	2.4	2.9	3.5	102	125	158	179	210
	9			0.7	2.4	3.1	3.6	4.1	110	158	186	206	236
1:2½:5	8½	Pumicite	10	0.5	1.8	2.5	3.0	3.6	100	115	138	160	196
	9			0.6	1.9	3.1	3.5	4.0	102	138	172	196	226
	9½			1.0	2.5	3.4	3.7	4.1	125	166	195	214	240

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In. Drop													
1:1½:3	5½	Portland	2	0.9	2.1	2.9	3.4	3.4	101	125	151	171	208
	6	Cement		1.3	2.9	3.6	3.9	4.1	133	172	198	216	238
	6½			1.7	3.4	3.8	4.0	4.2	160	199	219	234	251
1:1½:3	5½	Portland	5	0.7	2.0	2.8	3.1	3.7	100	121	140	160	190
	6	Cement		1.1	2.6	3.3	3.7	4.0	120	161	187	205	228
	6½			1.7	3.3	3.7	4.0	4.1	156	197	219	234	255
1:1½:3	5½	Portland	10	0.7	1.8	2.6	3.0	3.6	101	118	136	152	177
	6	Cement		1.1	2.6	3.3	3.7	4.0	111	151	178	199	223
	6½			1.8	3.3	3.9	4.0	4.3	153	192	216	232	250
1:1½:3	6	Portland	20	0.8	2.2	2.9	3.3	3.8	102	127	151	170	200
	6½	Cement		1.5	3.0	3.5	3.9	4.1	132	172	195	211	231
	7			1.8	3.4	3.8	4.0	4.3	166	205	227	239	256
1:2:4	6½	Portland	2	0.3	1.4	2.1	3.0	3.5	100	115	138	154	187
	7	Cement		0.7	2.2	3.0	3.6	4.0	104	140	169	195	224
	7½			0.9	2.7	3.5	3.8	4.1	131	176	203	219	243
1:2:4	6½	Portland	5	0.5	1.7	2.0	2.8	3.4	100	115	133	151	180
	7	Cement		0.7	2.1	2.9	3.3	3.9	101	131	160	183	213
	7½			0.9	2.5	3.3	3.8	4.0	130	167	193	213	238
1:2:4	6½	Portland	10	0.5	1.6	2.3	2.9	3.4	100	115	139	144	173
	7	Cement		0.9	2.2	3.0	3.5	4.0	101	137	168	191	221
	7½			1.0	2.8	3.5	3.8	4.1	129	171	195	215	238
1:2:4	7	Portland	20	0.6	1.9	2.7	3.1	3.6	100	122	145	164	198
	7½	Cement		1.0	2.6	3.1	3.7	4.0	110	152	182	202	229
	8			1.4	3.1	3.8	3.9	4.2	147	188	213	230	248
1:2½:5	8	Portland	2	0.5	1.4	2.4	2.9	3.6	100	121	146	173	205
	8½	Cement		0.5	2.0	2.6	3.1	3.9	112	146	176	196	225
	9			0.6	2.2	3.0	3.3	3.9	128	162	195	214	239
1:2½:5	8	Portland	5	0.5	1.6	2.3	2.8	3.5	100	116	146	171	205
	8½	Cement		0.5	1.7	2.5	3.1	3.8	104	139	170	191	222
	9			0.6	2.2	2.9	3.4	4.0	125	168	193	208	237
1:2½:5	8	Portland	10	0.5	1.8	2.6	3.0	3.8	100	117	143	164	198
	8½	Cement		0.6	1.9	2.7	3.3	3.9	106	139	173	195	222
	9			0.6	2.2	3.1	3.6	4.1	131	161	189	212	237
1:2½:5	8	Portland	20	0.3	1.4	2.1	2.6	3.2	100	112	132	148	182
	8½	Cement		0.6	2.2	2.9	3.4	4.0	112	145	176	199	228
	9			0.9	2.5	3.3	3.6	4.0	117	159	187	210	233
1:1½:3	5½	Hydrated	2	0.6	2.0	2.7	3.1	3.6	100	120	138	156	185
	6	Lime		1.0	2.5	3.2	3.6	4.0	111	150	179	199	228
	6½			1.3	3.1	3.7	3.9	4.1	147	190	214	231	252
1:1½:3	5½	Hydrated	5	0.4	1.5	2.2	2.6	3.2	100	115	127	138	164
	6	Lime		0.8	2.0	3.1	3.5	3.9	102	136	164	185	213
	6½			1.0	2.9	3.6	3.9	4.1	129	177	206	223	246
1:1½:3	6	Hydrated	10	0.6	1.9	2.5	3.0	3.5	100	120	135	150	175
	6½	Lime		1.0	2.5	3.2	3.5	4.0	103	138	166	188	214
	7			1.2	2.8	3.5	3.9	4.0	119	163	190	207	229
1:2:4	6½	Hydrated	2	0.3	1.3	2.0	2.2	3.2	100	111	125	141	170
	7	Lime		0.6	2.0	2.9	3.3	3.9	100	125	155	181	219
	7½			0.9	2.6	3.3	3.8	4.0	129	169	197	214	236
1:2:4	7	Hydrated	5	0.5	1.9	2.8	3.3	3.8	101	119	146	169	202
	7½	Lime		0.9	2.5	3.3	3.7	4.0	107	150	185	204	229
	8			1.2	3.0	3.6	3.9	4.1	132	182	210	227	248

TABLE 2—DATA OF SLUMP AND FLOW TESTS USING FLOW CONE—CONTINUED

Mix	Water Gal. Per Sack	Admixture		Flattening of Flow Specimens In. (Drops)					Flow—% (Drops)				
		Type	% By Wt.	0	5	10	15	25	0	5	10	15	25
Flow Cone—½-In. Drop													
1:2:4	7½	Hydrated	10	0.7	2.2	2.9	3.3	3.9	101	128	158	177	207
	8	Lime		0.9	2.7	3.5	3.7	4.0	106	156	188	202	229
	8½			1.2	2.9	3.5	3.9	4.0	130	176	204	218	239
1:2½:5	8	Hydrated	2	0.5	1.8	2.5	3.1	3.5	100	120	146	166	199
	8½	Lime		0.5	1.9	2.8	3.4	4.0	101	131	164	193	225
	9			0.7	2.2	3.1	3.6	4.0	124	153	190	211	238
1:2½:5	8	Hydrated	5	0.4	1.5	2.3	2.8	3.5	100	115	137	160	196
	8½	Lime		0.6	2.1	3.0	3.4	3.9	100	130	161	187	219
	9			0.5	2.3	3.1	3.5	4.0	115	157	188	210	231
1:2½:5	8½	Hydrated	10	0.6	1.9	2.7	3.2	3.9	100	122	147	165	201
	9	Lime		0.7	2.3	3.1	3.6	4.0	100	140	177	200	227
	9½			0.7	2.4	3.3	3.6	4.1	111	155	188	209	236

Current Researches on Plain and Reinforced Concrete and Related Materials

Report of Committee 101, Survey of Current Research

H. F. GONNERMAN, CHAIRMAN

INTRODUCTION

With the reorganization of the standing committees of the American Concrete Institute, Committee E-3, Research, was replaced by Committee 101, Survey of Current Research. With the change in the committee's name, a change was also made in its function. Those members of the Institute who are familiar with the reports of the old committee will recall that about half of the report was devoted to brief abstracts on researches published in current American and foreign engineering journals. The interest shown in these brief abstracts was one of the considerations, when Journal publication was inaugurated, in the creation of a section of Abstracts, and the formation of the new Committee on Abstracts, whose efforts would be entirely devoted to this type of work, giving much greater scope to the undertaking. Committee 101 will no longer include bibliographies in its report, but will devote its time entirely to assembling and compiling information on researches received from various college, state and commercial laboratories throughout the country. To this end, the subject-matter of this report has been prepared and is arranged in the following sections:

1—*Researches on cement.* Most of the researches already carried out or now in progress on cement are concerned with the constitution of portland cement and means of improving it, as well as with tests on high early strength cements. In addition, reference is made to the development of a comparatively inexpensive damp closet for cement testing.

2—*Researches on aggregates.* This section includes tests on durability of aggregates, effect of size and shape of aggregate particles, effect of heating aggregates prior to their incorporation in the mix, effect of grading, and the development of a comparative strength test for sand.

3—*Researches on plain concrete.* In the field of plain concrete the trend of the investigations is toward comparative tests on methods of curing, durability of concrete under various conditions of natural exposure, effect of use of various admixtures, and factors affecting volume changes. In addition, short summaries are given on tests of bricklaying problems, fatigue, flow under sustained stress, jigging, mixer operation, modulus of elasticity and Poisson's ratio, pavements, permeability and methods of obtaining watertightness, cast stone, compressive and flexural strength, temperature effects, uniformity of concrete test methods, wear, and workability.

4—*Researches on reinforced concrete.* The work on reinforced concrete includes studies on bond resistance of reinforced concrete beams under continued load, tests on bridge arches, and studies on type of reinforcing. An important investigation on columns is being carried out under the auspices of Committee 105 of the American Concrete Institute.

5—*Suggested researches on concrete and related subjects.* The number of researches suggested as needed and of prime importance indicates that there is a never-ending source of problems requiring study in the efficient use of concrete. These will be of particular interest to laboratories seeking subjects on which to direct their efforts.

This report has been submitted to letter ballot of the Committee, which consists of 11 members, of whom seven have voted affirmatively, none negatively, and four have refrained from voting.

H. F. GONNERMAN, *Chairman.*
F. E. RICHART, *Secretary.*

(NOTE—Many of the researches listed have not been completed, and in general, reports giving data of the tests are not available. In a few instances reference is made to the publication of reports. The conclusions given for certain of the investigations are those of the investigator and do not necessarily represent the views of the Committee.)

1—Cement

Improvement of Portland Cement: (Board of Water Supply, New York City). Studies in progress since 1914 with a view to improving the quality of portland cement, have covered such subjects as: Solubility in sugar solutions, heat of setting, alkali content, resistance to sodium sulfate solutions, lime-silica ratios, tests of cements previously used, and effects of calcium chloride and other salts. Results from these investigations are published from time to time in various engineering journals.

Stabilization of Di-Calcium Silicate: (Ohio State University, Columbus). Studies were made in the Department of Mineralogy on the properties of di-calcium silicate, particularly as they relate to refractories, but also in their application to portland cement products in the question of dusting. The results will be published at an early date by the Engineering Experiment Station.

The Upper SO_3 Limit in Portland Cement: (Mellon Institute, University of Pittsburgh). Only enough SO_3 should be added to portland cement to control its initial set, owing to the crystallizing tendency of the sulfoaluminate formed. Compressive strengths of small specimens of 1:3 mix, made from high early strength cement flour and Ottawa standard sand were about 5500 pounds per square inch at 24 hours and then rose steadily to 9200 at one year. As the great bulk of the cement was hydrated within the first day the increase strength must be ascribed to the induration reactions occurring within the silica gel.

Specifications and Tests for High Early Strength Cements: (State of Washington, Department of Highways, Olympia). The purpose of this investigation is to develop suitable specifications for the use of high-early strength cements. Tests were made on various kinds of cements at various ages using 6 by 12-in. cylinders, beams, and small mortar specimens. The data obtained thus far indicate that standard Ottawa sand briquets are unsatisfactory test specimens.

Development of a Low-Priced Damp Closet for Cement Testing: (Lehigh Portland Cement Co., Allentown, Pa.). Urgent need of an efficient low-priced damp closet for cement testing has led to a rather extensive search for simple devices for control of temperature and humidity. The most promising combination so far developed consists of a household refrigerator of the "top-icer" type, modified as to interior construction to provide a storage tank and shelves for pats and briquets; a nozzle of the oil spray type for discharging a water spray to provide humidification close to saturation, temperature control, and continuous circulation of the air inside the closet; and a thermostatic element to control the temperature of the water supply to the spray. While the limitations of this combination are not yet fully established, the object in view is to provide a cabinet that will automatically insure the conditions prescribed by the cement specification at all seasons of the year at a cost not to exceed $200 for the complete installation.

Slide-Rule for Calculating Compounds in Portland Cement: (Lehigh Portland Cement Co., Allentown, Pa.). In an effort to simplify the computation

of compound composition of portland cement, it was found that the percentage of any compound can be calculated directly from the analysis without the previous calculation of the percentage of any other compound. The series of equations for direct computation have been used as a basis for the design of a slide-rule for calculating the compound composition of portland cement. The slide-rule is described and illustrated in an article by L. A. Dahl, "A Slide-Rule for Calculating Compounds in Portland Cement," *Rock Products*, November 9, 1929, p. 50.

High Early Strength Concrete: (Pennsylvania Department of Highways, Harrisburg). This investigation was undertaken to determine the efficiency of high early strength cement compared with blended standard portland cement. Cement-sand mortar and concrete beams and cylinders were molded for test at age of 1 day to 1 year. Tension and compression tests were made on the cement sand mortars, beams were tested for modulus of rupture, and cylinders were tested in compression. The special cement showed increased strength at ages of 1 to 28 days, but at the 3-month period the strengths were nearly equal in the mortar tests and in the modulus of rupture tests on concrete. The concrete compression tests showed the special cements to be slightly higher than the standard portland.

In another series of tests which has just been completed, a high-early strength cement showed a considerable gain in mortar tension and compression for the first 7 days. At the 28-day period and later periods, the strengths converged, and in some cases the standard cement showed slightly higher strengths. In the concrete tests the high early strength cement showed slightly higher strengths at the 1- and 2-day periods, but strengths thereafter were about equal to those of the standard portland cement.

Comparative Strength Tests of Cement: (Iowa State Highway Commission, Dept. of Materials and Tests, Ames). Tests were carried out in cooperation with Sub-Committee VII of A. S. T. M. Committee C-1 on Cement and the Special Committee on Cement of the American Society of Civil Engineers, and the results obtained were forwarded to the respective committees for comparison with the results obtained by the other cooperating laboratories.

Study of the Effect of Quality of Cement on Quality of Concrete: (U. S. Bureau of Public Roads, Washington, D. C.). 3-year tests have been completed and report will be issued shortly.

Researches on Cement: (Portland Cement Association Fellowship, National Bureau of Standards, Washington, D. C.) The investigations of this Fellowship are designed to explore the major chemical problems of the cement industry. Among the investigations now in progress are the following:

(1) A study, with pure materials, of the influence of composition on the volume constancy of specimens of the set cements. The compositions examined include the cement compounds, mixtures of compounds, laboratory cements of systematically varying composition prepared under definitely controlled conditions, and commercial cements. The specimens, 6 by 1 by 1-in. are stored (a) continuously in air, (b) continuously in water after the

first 24 hours, and (c) alternately in air and in water at 7-day intervals. Length measurements are made at definite time intervals, over a period of years, using a microscope comparator and a micrometer comparator.

(2) A comprehensive study of the set of cements which includes research on (1), the chemical phenomenon of set; (2), the influence of rate of set on the properties of the cement; (3) the measurement of rate of set; and (4) the control of set. During the past year some of the causes have been examined of the so-called "false" or "hesitating" set. Also, a study is under way on the heat of setting and a correlation of the heat evolved with the reactions taking place as determined by microscopic, chemical and physical tests.

(3) A fundamental investigation on the kinetics and thermodynamics of the hydration or setting process. The rate and degree of hydration are being measured by observations on the change in density of samples of the cement compounds of known specific surface, and by observations on the heat of hydration of the compounds. Likewise, a thermal analysis of the hydrated compounds is being made by the continuous weighing of samples undergoing dehydration under rigidly controlled conditions.

(4) Phase equilibria study of the cement systems. During the past year the system CaO-Al_2O_3-Na_2O has been studied. Soda is commonly present in cements in small amounts and it is important to have information on the compounds which it forms. Only then can a rational study be made of the influence of the soda under all conditions.

(5) The development of the x-ray method to a study of the constitution of portland cement. It often is not possible by microscopic or chemical means to identify the phases present in laboratory or commercial cements. The x-ray method has been developed to a point where it is of great value in the identification of compounds and in the following of reactions. Many of the problems which have long baffled investigators have yielded to the convincing criteria of the x-ray.

Recent publications of the Fellowship are as follows:

Paper No. 21—"Calculation of Compounds in Portland Cement," by R. H. Bogue, *Ind. Eng. Chem.* (Analytical Edition), I, (1929), 192; *Rock Products 32*, No. 23 (1929), p. 47.

Paper No. 22—"The Influence of Magnesia, Ferric Oxide and Soda upon the Temperature of Liquid Formation in Certain Portland Cement Mixtures," by W. C. Hansen. *Bureau of Standards J. of Research*, Jan., 1930, Research Paper No. 132.

Paper No. 23—"Revised Procedure for the Determination of Uncombined Lime in Portland Cement," by William Lerch and R. H. Bogue. *Ind. Eng. Chem.* (Analytical Edition) *2* (1930), 296; *Concrete, 37*, (1930), 88.

Paper No. 24—"The X-ray Method Applied to a Study of the Constitution of Portland Cement," by L. T. Brownmiller and R. H. Bogue. *Bureau of Standards J. of Research*, October (1930) R. P. No. 233, *Am. J. Sci.* (in press).

Researches in Cement: (A. S. T. M. Committee C-1 on Cement). At the meeting of Committee C-1 of the American Society for Testing Materials held in Washington, Sept. 6, 1930, the following sub-committees were formed:

"A"—Methods of Testing
"B"—Specifications for Portland Cement
"C"—Specifications for High Early Strength Cement
"D"—Specifications for Masonry Cement

These sub-committees are empowered to form technical committees for the purpose of studying specific problems. The technical committees are not to be considered as standing committees but are expected to complete their work and make a final report as early as possible.

Sub-Committee "A" will give its immediate attention to the rewriting of the Manual and will undertake to suggest such changes in present apparatus as may be necessary and also to develop better apparatus for cement testing than is available at present. The development of a moist closet which will maintain constant temperature and humidity is quite an urgent question as many laboratories are willing to install this equipment it a suitable one can be economically manufactured.

Sub-Committee B has appointed 3 technical committees to report on the following problems relating to portland cement:

1. Relation of Particle Size to Physical Properties,
2. Relation of Chemical Composition, particularly as Affected by Ratios, to Physical Properties,
3. Study of Accelerators and Retarders,
4. Effect of Temperatures at time of use,
5. Study of Accumulated Data on Plastic Mortar Strength Tests, and Recommendation as to Whether Work Shall be continued on this Subject,
6. Study of Tests Indicative of Durability,
7. Early-age Tests.
8. Factors Affecting Volume Change.

2—AGGREGATES

Effect of Mixing Coarse Screenings with Sand which Does Not Meet Specifications: (Kentucky State Highway Department and University of Kentucky, Lexington). This procedure has been in operation for two years, with excellent results. A complete report on the findings from this study will be published soon.

Durability of Concrete Made from Unsound Stone: (Kentucky State Highway Department and University of Kentucky, Lexington). This investigation included tests on concrete made with 50 and 100 per cent unsound fine and coarse aggregate. Concrete made with 100 per cent unsound aggregate and exposed to weather tested as follows: 3 per cent increase in strength from 28 days to 6 months, with no failures due to disintegration; 11 per cent decrease in strength from 6 months to 1 year, one specimen in three having failed through

disintegration; 44 per cent increase in strength from 1 year to 2 years, with two failures due to disintegration.

Concrete made with only 50 per cent unsound aggregate tested as follows: 36 per cent increase in strength from 28 days to 6 months; 13 per cent increase from 6 months to 1 year; 27 per cent increase from 1 to 2 years. In this group of tests there was some crazing but no disintegration. Tests will be made through the 5-year period.

Sandstone as Aggregate in Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). An extensive investigation has been in progress since 1920 to determine the suitability of sandstone as fine and coarse aggregate in the construction of concrete roads and structures. It was found that any sound sandstone having a percentage of wear of less than 7 can be used in concrete pavement under proper control methods. Sandstone with a percentage of wear as high as 15 can be used for structures. A report is being prepared for early publication.

Method of Testing Stability of Sandstone for First-Class Masonry Construction: (Kentucky State Highway Department and University of Kentucky, Lexington). Tests were made to determine the reliability of sodium sulfate soundness tests on sandstone common to Kentucky. Specimens which went to pieces under the accelerated test have not been affected after two years in the weather, and the same stone was not affected by twenty alternations of freezing and thawing. Complete chemical analyses are being made of the stone which passed the accelerated soundness test and on the samples which did not. Though indications are that this test is not suitable for this type of aggregate, no conclusions can be drawn until further tests are made.

Properties of Concrete Made with Light-Weight Aggregate: (T. & A. M. Dept., University of Illinois, Urbana). The purpose of this investigation was to determine the properties of a structural grade of Haydite concrete such as might be suitable for reinforced concrete buildings and other structures. Studies have been made thus far on the properties of the aggregate, as well as on compressive strength, weight, density, modulus of elasticity, and workability of concrete of varying proportions, richnesses, and consistencies. In addition, determinations are being made of the resistance to diagonal tension, bond and compressive stress in flexure of reinforced concrete beams. Data of the tests were given in paper on "Tests of Plain and Reinforced Haydite Concrete," by F. E. Richart and V. P. Jensen, Proceedings, American Society for Testing Materials, Vol. 29, 1930; also by the same authors, "Construction and Design Features of Haydite Concrete," this JOURNAL, A. C. I., Oct., 1930, p. 151: "Tests of Bonding Floor Finish to Slabs of Haydite and Sand Concrete," JOURNAL, A. C. I., Dec. 1930, p. 339.

Effect on Strength of Soft Particles of Coarse Aggregate on the Strength and Durability of Concrete. (Michigan State Highway Department Laboratory, Lansing). This investigation was undertaken to determine to what extent the strength of concrete is affected by the presence of shale, friable sandstone, ochre, etc. 6 by 12-in. cylinders and 6 by 6 by 40-in. beams were made from concrete in which an aggregate known to be sound was mixed with varying

percentages of soft and non-durable particles. In one group of tests, the specimens were broken after 28 days' moist curing, and in another group after 25 cycles of freezing and thawing following 28 days moist curing. The strength of concrete, both moist-cured and subjected to freezing and thawing, was reduced by the presence of these particles. Report on this investigation by W. J. Emmons published, Proceedings of the 16th Annual Conference on Highway Engineering, University of Michigan, p. 45.

Effect of Character of Fine Aggregate on Mortar Strength: (Minnesota Highway Department, St. Paul). Six sands from different pits in Minnesota were artificially graded and tested in mortar in compression, tension, and flexure. The results show that with grading and water-cement ratio constant, variation in shape and characteristics of the sands had practically no effect on the strength of the resultant mortar.

Effect of Heating Aggregates on Concrete Strength: (Minnesota Highway Department, St. Paul). Compression and transverse specimens made for test at 3, 7, 14, 28 and 180 days, 12, 18 and 24 mo., from sand, gravel, and water heated sufficiently to bring the temperature of the concrete to 70, 100 and 130 degrees F. when deposited in the molds. Time of set determined by flow. It appears from the data obtained thus far that: (1) heated concrete requires more water to produce a given workability, (2) there is a reduction in strength so great that the difference in water-cement ratio can hardly be the cause, (3) heated concrete sets more quickly than unheated, (4) concrete should be placed in forms at a temperature of not over 70 to 80 degrees F.

Joint Influence of Iron and Aluminum in Native Sands and Mortar Strengths: (Maine Technology Experiment Station, Orono). The status of this investigation was reported in the Proc. of National Acad. of Sciences, v. 15, p. 742, Sept., 1929, and reprinted as Paper 9 of the Maine Technology Experiment Station.

Influence of Aluminum on Mortar Strength: (Maine Technology Experiment Station, Orono). The status of this investigation was reported in the Proceedings of the National Academy of Sciences, v. 15, p. 740, Sept., 1929, and reprinted as Paper 8 of the Maine Technology Experiment Station.

Study of Hydrogen Ion Content of Maine Sands in Relation to Their Use in Concrete: (Maine Technology Experiment Station, Orono). It appears from the results obtained thus far that the hydrogen ion content will add much to the knowledge of strength-making qualities of sands.

Selection of Mortar and Plaster Sands: (Lehigh Portland Cement Company, Allentown, Pa.) Two comparatively new methods of test were employed in these studies, one of which is the so-called plastic mortar test which provides a means of judging sands from the point of view of economy, as well as a method for determining quantitatively the deleterious effects of organic matter in sands. The other was a simple titration method for determining the absorption of sand. The former test is under investigation by Committee C-1, of the American Society for Testing Materials as a possible specification test for portland cement and the latter test is useful in connection with all concrete tests in which the absorption of the fine aggregate is a factor. The

sand tests were described and discussed by J. C. Pearson in paper on "Fine Aggregate in Mortar and Plaster" (see Proc. A.S.T.M. 1929) and "A Simple Titration Method for Determining the Absorption of Fine Aggregate" (see *Rock Products*, p. 64, May 11, 1929).

The Laws of Grading Derived and Applied to Aggregates and Cement: (Mellon Institute, University of Pittsburgh and Bureau of Mines, Minneapolis Station). Fundamental laws have been derived by a mathematical analysis for maximum packing, giving two systems, one with an intermittent or gap grading and the other continuous. These have been applied to mortar, where noteworthy applications have been made to stucco, cast stone and masonry mortars, to concrete and to the proper distribution for the cement itself. The function of admixtures is analyzed.

Effect of Grading of Fine and Coarse Aggregate on Strength of Concrete: (National Sand and Gravel Association, Washington). This investigation was designed to furnish information on the effect of maximum size of sand and gravel on quality of concrete, and the effect of variations in grading within the usual specification limits. Tests were made in compression and flexure, using three different concrete mixes involving three gradings of fine aggregate and seven gradings of coarse aggregate.

Effect of Flat Particles on Concrete-Making Properties of Gravel: (National Sand and Gravel Association, Washington). The report on this investigation, which consisted of compressive and flexural tests of concrete of two different mixes using gravel free from flat particles and gravel containing 15 per cent flat particles, was published in the 1929 Proceedings of the American Society for Testing Materials, and in the July, 1929, issue of the National Sand and Gravel Bulletin.

Effect of Grading on Void Content: (National Sand and Gravel Association, Washington). This investigation supplements the work on this subject described in the 1928 Report of this Committee. The report on these tests, together with a report on the previous researches, has been published in the National Sand and Gravel Bulletin and as Circular 8 of the National Sand and Gravel Association, and includes data of tests on void content for all possible combinations of three sizes of sand—No. 14 to No. 4, No. 48 to 14, and 0 to No. 48—and for 57 combinations of four sizes of gravel—¾ to 1½-in., ⅜ to ¾-in., No. 4 to ⅜-in., and No. 8 to No. 4.

Effect of Mineral Composition and Shape of Particles in Coarse Aggregate on Strength of Concrete: (National Sand and Gravel Association, Washington, D. C.). This investigation is designed primarily to show what variations in quality of concrete may result from a difference in mineral composition of the coarse aggregate or from a difference in the shape of particle. It includes a large number of different aggregates from widely separated sources tested under comparable conditions. A feature of the tests is the attention paid to differences in the void contents of the different aggregates and the effect of this factor on the quantity of mortar required to produce workability. Both compression and flexural tests are being made.

Effect of a Amall Amount of Colloidal Clay in Sand on Its Usefulness as a Fine Aggregate. (University of Colorado, Boulder). A leading sand and gravel company of Denver has been having trouble in marketing the sand that is obtained in connection with their commercial gravel. The sand and gravel deposit underlies 2 or 3 feet of top soil and is very extensive and uniform. With nominal washing the gravel is entirely satisfactory, but although the sand makes concrete that is satisfactory from the standpoint of strength, contractors have objected that it is "dead" and does not work properly. It has been necessary, therefore, to waste most of the 0-4 material from this pit. After various tests it was found that the presence of a small amount of colloidal clay seemed to be the main source of the trouble. Some of this clay is so fine that water containing it will remain clouded for days. This slime is very tenacious and the adhesion is such as to impart a distinct crustiness to any dried surface of the unwashed sand. When the clay is washed out and dried it resembles hardened cement and has considerable strength in tension. A thorough washing removed all signs of the objectionable "deadness" and no crust formed upon drying. Special cone slime removers and washing equipment were installed as a result of these tests and the sand has apparently been rendered a satisfactory and marketable product. The washing also removed some indications of organic contamination as shown by the colorimetric test.

Effect of Briquet Molding Pressure on Strength Ratio of Sand Mortars: (National Sand and Gravel Association, Washington). This investigation was outlined for the purpose of obtaining information on the effect of different laboratory practices on results of tests of sands in mortar. Preliminary studies indicate that the pressure applied in molding briquets may affect the strength of concrete sand mortars differently from standard sand mortars, and hence may affect the strength-ratio.

Crushing and Abrasion Tests of Gravel: (National Sand and Gravel Association, Washington). In a study of quality of gravel particles, crushing tests and modified Deval abrasion tests of a large number of samples of gravel are being made. The crushing test is being made by applying a pressure of 3000 lb. per sq. in. on a 3 kg. sample of gravel, graded 50 per cent 1 to 1½-in., 25 per cent ¾ to 1-in., and 25 per cent ½ to ¾-in., confined in a cast iron cylinder 6 in. in diameter. The reduction in fineness modulus and in volume of voids due to application of load is being determined. The abrasion test is being made on a sample of the same grading, to which 6 cast iron balls 1⅞-in. in diameter are added as an abrasive charge, revolved 10,000 revolutions in a standard Deval cylinder at 33 r. p. m. These data are included in a progress report to Sub-Committee 2 of Committee 2 on Ballast of the American Railway Engineering Association (Bulletin No. 321, Vol. 31, Nov., 1929, p. 761).

Accelerated Soundness Tests of Gravel: (National Sand and Gravel Association, Washington). Preliminary tests have been carried out to determine whether the difference in abrasion test results of gravel before and after treatment with sodium sulfate, in accordance with the accelerated soundness test, may be used as an indication of the soundness of the material. Results

on gravels studied to date have been unsatisfactory, since a greater resistance to abrasion after treatment with sodium sulfate is shown sometimes than was obtained on the untreated samples.

Durability Studies of Aggregates for Concrete: (Research Laboratory, Portland Cement Association, Chicago). Comprehensive studies are being made of the variations in aggregate quality that may be encountered in pits and quarries and the effect of such variations on the durability of concrete in which the aggregates are used. The study embraces petrographic and geologic studies in the field and laboratory, sodium sulfate tests for soundness of aggregate, freezing and thawing tests of the aggregates and of concretes and mortars in which the aggregates are incorporated. A major objective of this study is the establishment of some physical test by which the suitability of aggregate for use in concrete can be determined.

Well-Graded Washed Sand Versus Crushed Granite Screenings as Fine Aggregate: (University of Colorado, Boulder). For the conditions of these tests the granite screenings gave concretes which were decidedly above those from the washed sand in strength and fully equal to them in workability. These tests were with identical coarse aggregates and water-cement ratios. Mixtures of sand and screenings gave strengths below those for the screenings alone but above those for the washed sand.

Limestone and Gravel Used as Coarse Aggregate: (Pennsylvania Department of Highways, Harrisburg). Tests were made to compare the relationship between strength of concrete in which limestone and gravel were used as coarse aggregate. Modulus of rupture tests were made on concrete beams and compression tests on concrete cylinders at ages of 1 day to 1 year. At all ages the crushed limestone gave higher strengths than the gravel. The modulus of rupture of the limestone concrete at various ages ranged from 10 to 20 per cent higher than that of the gravel concrete. In compression, the limestone concrete was between 20 and 35 per cent higher than the gravel concrete.

Use of Soft Limestone in Concrete: (Iowa State Highway Commission, Ames.) Tests were made to determine the effect on strength and durability of concrete of the use of relatively soft limestone as coarse aggregate. Six by 6 by 30-in. beams and 6 by 12-in. cylinders were made using normal limestone in one group, and a very soft limestone in companion specimens. Strength tests indicate a slight decrease in both flexural and compressive strengths particularly for soft limestone which was found to be unsound in the sodium sulfate soundness test.

Development of a New Strength Test for Sand: (Iowa State Highway Commission, Ames). The purpose of this investigation was twofold: (1) to develop a strength test for sand which will accurately and consistently evaluate the concrete-making ability of different sands, (2) to develop a method of designing concrete mixes on the basis of this test. At various periods for the past 9 years work has been undertaken on these problems using different phases, different approaches, and different ideas. Of all the methods tried, the mortar-

voids method of Talbot and Richart seems to be the most dependable, the nearest approach to the realization of the basic laws which govern the strength of concrete. During the past year, several thousand 2 by 4-in. and 6 by 12-in. cylinders and 6 by 6 by 30 and 2 by 2 by 14-in. beams were made and tested. The method of Talbot and Richart of designing mixes was tried out in the field and used in the construction of about 200 miles of pavement. Tests were made on core specimens from this pavement and on beams made during the construction of the pavement. The technique of Talbot and Richart has been improved upon to such an extent that some of the difficulties they met with have been eliminated, as well as new difficulties which arose during these tests. The technique has now reached the point where the effect of such variables as amount of water required to wet the surface of coarse aggregate variations in manner of mixing and placing, and workability as distinguished from consistency are factors of prime importance. When these factors have been evaluated, it will be possible to design concrete mixes for a definite strength and attain values within a range of considerably less than 10 per cent of the desired result. In addition, it will be possible to estimate accurately quantities of all materials after a few hours' work with the sand and cement to be used in the concrete. The attainment of this is, of course, based on the assumption that the curing condition will be the same, and that the coarse aggregate is structurally sound and of the quality normally used.

Short-Time Strength Tests for Concrete Sands: (Kansas State Agricultural College, Manhattan). This investigation was undertaken in 1925 to determine the relation between the strength-ratios of various natural and commercially available sands in the State, using a high early strength cement and those obtained using portland cement in the ordinary 7 and 28-day strength tests.

The procedure is to test a number of sets of three briquets and three 2 by 4-in. cylinders made with each sand and each high early strength cement at 24 and 48 hours, for comparison with corresponding sets of briquets and cylinders made on the same day from standard Ottawa sand and the same high early strength cements. Similarly 7 and 28-day strength ratios will be obtained for each of the sands with ordinary portland cement. It should be possible to secure certain coefficients for interpreting 24 and 48-hour strength ratios in terms of 7 and 28-day strength ratios. Also, it is possible that curves may be obtained which will show directly the relationship between strength ratios of early strength cements and those of ordinary portland cement with the various sands available in Kansas.

Study of Effect of Character of Fine Aggregate on Flexural Strength of Mortars: (U. S. Bureau of Public Roads, Washington). This investigation has been undertaken to determine the importance of character of fine aggregate on resultant strength of mortar in flexure.

Study of Methods of Determining Absorption and Free Water Content of Fine Aggregates: (U. S. Bureau of Public Roads, Washington). This group of tests is being made to develop, if possible, efficient methods for determining absorption and free water content of fine aggregates.

Study of Methods of Making Abrasion Tests on Fine Aggregates and Hardness Tests of Mortars: (U. S. Bureau of Public Roads, Washington). This is a continuing investigation to study the relation between abrasion tests on fine aggregates and hardness tests of mortars.

Studies of Effect of Type of Aggregate on Resistance of Concrete to Frost Action: (U. S. Bureau of Public Roads, Washington). This investigation has been completed, and a report on the results obtained will be published shortly.

Study of Relation between Coarse Aggregate Content and Quality of Pavement Concrete: (U. S. Bureau of Public Roads, Washington). The object of this investigation was to determine the effect of variations in the amount of coarse aggregate on workability and quality of portland cement concrete in pavement slabs constructed with standard equipment. The study involves the construction of approximately one-half mile of concrete pavement 9 feet wide and 7 inches thick, constructed in 9-ft. sections. Both crushed stone and gravel were used as coarse aggregate and the proportions varied from a distinctly over-sanded mix (1:2:3½) to mixes containing as high as 4¾ parts of crushed stone and 5¼ parts of gravel. Observations were made on the relative workability of the various concretes when finished with both of the standard finishing machines now on the market. The quality of the concrete was determined by flexural tests on concrete beams 27 in. wide and 5 ft. long taken from the test sections. Other tests for quality, such as compression tests on cores drilled from the slabs, and freezing and thawing tests of concrete specimens taken from the beams, were also made.

Investigation on the Quality of Concrete for Bridge Floor Slabs with Special Reference to Use of Light-weight Aggregates: (U. S. Bureau of Public Roads, Washington). This investigation is under way in cooperation with the Port of New York Authority.

3—Plain concrete

The researches under way on plain concrete which are described below have been listed under the following classifications:

Admixtures
Bricklaying
Crazing
Curing
Dams
Durability
Fatigue
Fire Resistance
Flow under Load
Jigging
Mixers and Mixing
Modulus of Elasticity and Poisson's Ratio
Pavements
Permeability
Pipe
Products
Strength—compressive, flexural
Temperature Effects
Tests and Test Methods
Volume Changes
Wear
Workability

ADMIXTURES

Effect of Sugar on Strength of Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). Tests were made on sand, cement, and coarse aggregate which were sent to the Laboratory in sugar containers, such as sacks and syrup buckets. It was found that very small amounts of sugar reduce the strength of mortar to almost nothing, and that very unreliable results are obtained on material shipped in such containers. Fine and coarse aggregate shipped in this manner, however, tested satisfactorily after being thoroughly washed. Results of this investigation were published in Kentucky Highways, v. 3, No. 3, Sept., 1928.

Tests of Admixtures: (California Division of Highways, Sacramento). Tests are under way to determine the effect of additions of various commercial powders on the properties of cement mortars and concretes. Laboratory compression tests are being made on 2 by 4-in. mortar specimens and on cores drilled from pavements. Results not yet available.

Celite as an Admixture: (University of Colorado, Boulder). The strengths of concretes and mortars with 1:2 and 1:3 cement-sand ratios were increased appreciably by additions of Celite of 3, 5 or 10 per cent of the weight of the cement. The mixtures were stiffened by the Celite and at constant workability the required increase in water-cement ratio offset the effect of the Celite. Both fine and coarse aggregates were crushed granite of excellent quality. While the mixtures were not harsh prior to the addition of Celite there was apparent the usual softening or "fine sand" effect. (In some former tests with different cement and aggregates, there was no apparent strength difference between concretes and mortars with and without Celite at constant water-cement ratio). Celite had no measurable effect on yield in either series.

Permeability Tests of Concrete with and without Admixtures of Celite: (Pierce Testing Laboratories, Denver, Colorado, for Johns-Manville Corp.). Permeability tests were made on 1:2:4 concrete mixtures, with and without admixtures of Celite, subjected to water and oil at pressures up to 200 lb. per sq. in. In a 1:2:4 mix with a water-cement ratio of 0.85, the slump for concrete without Celite was 4 in., and that with the admixture in amounts of 3 lb. per bag of cement, the slump was 1.75 in. The compressive strength at 7 days was 2645 and 2910 lb. per sq. in., and at 28 days was 4017 and 4652, for untreated and treated concrete, respectively.

The concretes were cast into 14 by 14-in. concrete cylinders, with a pipe projecting from the center, at the bottom of which was left a cavity 1½ by 1½ in. thus leaving a wall of concrete 6 in. thick in every direction, on which the pressure was applied. These specimens were connected with the apparatus, each having a reservoir equipped with a water or oil gage, thus making it possible to read the amount of water or oil forced into or through each specimen at any time. These reservoirs in turn were connected to an air tank equipped with pressure gage, air compressor, and motor by means of which the pressure was raised to and maintained at the desired amounts. On total

loss of oil, Celite concrete showed a permeability 15.4 with plain concrete considered as 100; on total loss of water, Celite concrete showed a permeability of 29.5 on this same basis.

Effect of Calcium Chloride on Hydration of Minerals: (Ohio State University) This investigation was conducted in the Department of Mineralogy in cooperation with the Calcium Chloride Association. The results obtained will be published by the Engineering Experiment Station.

Effect of Various Powdered Admixtures on Workability of Concrete: (Missouri State Highway Department, Jefferson City). These tests were made to determine the efficacy of various powdered admixtures for improving the workability of concrete. Laboratory tests were made with the Smith-Conahey workability apparatus on concrete containing different admixtures in varying amounts, all batches being gaged with enough water to give a constant flow. The basic mixture used was 1:2:3½ at a flow of 125 (15⅛-in. drops). Two sands were included, one a rounded-grain quartz river sand, and the other an angular-grained crushed flint sand. Field tests and observations were also made of the use of these admixtures in actual construction. Though the investigation has been completed, no report is available for publication.

Effect of Crusher Dust on the Strength and Other Properties of Mortars and Concretes Made from Crushed Granite: (Univ. of Colorado, Boulder). Crusher dust (of the same material as the aggregates 0-100 grading) was added to mortars and concretes as a check on the extent to which accidental concentrations of dust might be harmful to the quality of the concrete. At a constant water-cement ratio the dust strengthened the concrete but stiffened it materially. At constant slump the increase in strength from adding dust was about equal to the decrease from increasing the water-cement ratio up to an amount of dust equal to 15 or 20 per cent of the total fine aggregate. The conclusion from these tests and materials is that no ordinary concentration should be injurious but it may complicate the control of mixing water and workability. The results from these tests should not be extended to other conditions and materials without experimental verification, however.

Use of Diatomaceous Earth in Concrete. (Pennsylvania Department of Highways, Harrisburg). This investigation was undertaken to determine the effect of admixtures of diatomaceous earth on the properties of concrete. Concrete cylinders and beams were molded using the same water-cement ratio and same materials, except that diatomaceous earth was added to one group of specimens in amount of 3 lb. per bag of cement. Concrete containing the admixture was found to be more plastic, and the surface of the specimens smoother, while the plain concrete had a higher modulus of rupture and higher compressive strength. The admixture did not reduce the absorption of the concrete.

Use of Calcium Chloride in Concrete: (Iowa State Highway Commission, Ames). The purpose of this investigation was to determine the effect of calcium chloride used as an admixture in concrete. Two series of 6 by 6 by

30-in. beam specimens were prepared, one set during the construction of a paving project which was started in 1926, and the other set in the laboratory. For the field series the principal variables were curing condition and type of aggregate. For the laboratory specimens a number of variables was introduced, such as different brands of cement, various types of coarse aggregates, and different consistencies. A final report will be prepared supplementing the progress reports of January and August, 1927.

Use of Admixtures in Concrete: (Iowa State Highway Commission, Ames). These tests were undertaken to determine the effect of various inert admixtures in concrete, including pumicite, hydrated lime, Celite, Morrisite, and portland cement. Specimens tested were 6 by 6 by 30-in. beams and 6 by 12-in. concrete cylinders containing varying amounts of the admixtures. The strength tests obtained to date indicate little or no advantage in using admixtures in the amounts recommended by the producers. Some of them, particularly Celite, increase the yield slightly, and somewhat out of proportion to the absolute volume of the material added. This appears to be due to the additional water required. Workability, the talking point of the producers, is as yet unmeasured by any means which affords a common measure for concrete made with different admixtures.

Tests of Cements and Admixtures: (Delaware, Lackawanna, and Western Railway Co., Hoboken, N. J.). 6 by 12-in. cylinders were made for test at 7, 28, 60, 180, and 360 days from concrete with and without 5 per cent admixtures by weight of cement of Celite, Cal, Dowflake, Colloy and Fluresit, using two standard portland cements as well as Lumnite and trass cements. Two water-cement ratios were used for each group—6.25 and 6.75 gal. per sack—the aggregates were dry, and the slump was 3 in. in all tests. In tests planned for future work Incor cement will be included to determine its early strength quality and its strength up to one year.

BRICKLAYING

Investigation on Bricklaying Problems: (Mellon Institute, University of Pittsburgh, Pittsburgh). This series of tests was undertaken by Mellon Institute, in collaboration with the Eastern Face Brick Manufacturers' Association, to study those combinations of variables representing the probable range of practical application. About 400 experimental walls and panels were erected in the grounds of Mellon Institute Building. Since the most obvious point of attack are studies of the characteristics of different kinds of brick and mortar, the brick and mortar problems under investigation include the absorption and surface characteristics of brick, and differences in mortars due to varying the cementing materials, sand, and pigments. Properties of mortars being studied are workability, compressive and transverse strength, absorption and permeability, shrinkage, durability, staining and efflorescence, and elasticity. Information is also being collected to determine whether the 10 per cent lime often specified by architects, or the 25 per cent or more usually employed by brick masons, gives better results.

The problems in backing include a study of hollow tile of different sizes, of brick, of cinder or concrete block, of brick tile, and of metal or other lath on

steel frame. Variations in workmanship are most apparent in regard to tapping, pointing, and the filling of head-joints. Tapping is being studied not only with relation to the effect of excessive downward tapping into place, but also with relation to plumbing both before and after initial set.

Special mention should be made of weather problems. The experiments are planned for the purpose of investigating the behavior of mortar with reference to the other variables in all types of climatic conditions. Summarized recommendations as well as the complete report are being published by the American Face Brick Association.

CRAZING

Development of Hair Cracks into Structural Cracks: (University of Michigan, Ann Arbor). This investigation was started in 1926 to determine whether hair cracks develop into structural cracks on alternate wetting and drying of neat cements and rich mortars. Though the data are being obtained very slowly, it appears that hair cracks become larger and deeper with alternations of volume change.

CURING

Rates of Evaporation and Absorption of Mortar and Concrete Specimens of Different Sizes and Shapes: (University of Colorado, Boulder, in cooperation with the U. S. Bureau of Reclamation and the Arch Dam Committee of Engineering Foundation). By frequently weighing large numbers of specimens of different sizes and shapes as they were exposed to the air after different periods of curing in water a very large quantity of data on the rates of moisture loss have been collected. Among the sizes and shapes of specimens were cylinders (2 by 4-in., 3 by 6-in., 3 by 12-in., 3 by 18-in., 4 by 14-in., and 6 by 12-in.) and beams (3 by 3 by 40-in., and 4 by 6 by 38-in.). It is reasonable to expect the rate of moisture loss to be more or less proportional to the surface volume ratio for a given mixture, period of moist curing, and type of exposure. This presumption was borne out in part. Other tests were made to ascertain the absorption rates upon re-immersion. The absorption rate is much higher than the rate of moisture loss, very substantial quantities of water being taken up by dry concrete during the first few seconds of immersion. These data are in process of preparation for publication.

Curing Concrete Pavements: (Kentucky State Highway Department and University of Kentucky, Lexington). The purpose of this investigation was to determine whether pavement protected with wet burlap for 24 hours is as satisfactory as that covered on the surface with calcium chloride following the 24-hour wet burlap curing. The concrete used was a 1:2:3.5 mix by volume field condition, with an average cement factor of 1.52. Washed bank sand and gravel of satisfactory gradation were used with standard portland cement averaging about 300 lb. in tensile strength at 7 days. Two 7 by 7 by 30-in. beams were made each day for 20 days; both were covered with wet burlap for the first 24 hours, after which one beam of each pair was surface-treated with calcium chloride. The specimens were tested in flexure and compression

at 7, 14, 21 and 28 days. During the construction of the pavement the concrete was covered over with wet burlap for the first 24 hours, after which every alternate 50-ft. section was treated with calcium chloride. Two cores were taken from each treated and untreated section. The results obtained from these tests indicate that there is no advantage in using calcium chloride, since almost equal strengths were obtained for both treated and untreated concrete, and the untreated concrete developed sufficient strength at 7 days both in flexure and compression to permit opening the pavement. A study of cracking of the pavement did not show any advantage of the treated over the untreated concrete.

When the project was nearing completion it was found that the supply of calcium chloride was exhausted. It was decided to omit calcium chloride from the last 3000 ft. of pavement, but to treat 2000 ft. of concrete with silicate of soda. The small amount of work done in this connection indicated that there is nothing to be gained by using this material.

Imperial Valley Curing Investigation: (California Division of Highways, Sacramento.) An extensive investigation is under way to determine the efficacy of various methods of curing concrete pavements. Bending tests are being made on beams cast in pavement and on cores drilled from the concrete. Tests made up to and including 28 days on all curing methods showed lower strengths than standard water curing. At 3 months, three of the methods gave slightly higher strengths.

Effect of Methods of Curing on Strength and Wear Resistance of Concrete: (Texas A. & M. College, College Station). Tests are being made to determine the strength and resistance to wear of 250 beams and 500 cylinders cured by means of calcium chloride, Hunt process, curcrete, silicate of soda, wet earth, regulated sprinkling, and no curing.

Curing of Concrete: (Ohio State University, Columbus). Researches were made to determine the effect of calcium chloride as a curing agent for portland cement concrete. These studies were made as a part of a graduate thesis, in cooperation with the Calcium Chloride Association, and will be published by the Engineering Experiment Station.

Comparison of Methods of Curing Concrete Pavements: (Wisconsin Highway Commission, Madison). This investigation covered comparative tests of various methods of curing, using wet earth curing as the criterion. The tests were run in 3 series of five 400-ft. curing sections. One-half the slab was cured with wet earth, and a wood header was placed down the center joint and tarred thereto to prevent seepage. The other half was treated with the various methods included in these studies. Three sets of beams of two each, one earth-cured and the other with special curing, were cast within road forms, and hand-finished composite specimens were made outside road forms. In addition, cores were taken at various ages.

Loss of Moisture from Concrete Cured by Various Methods: (Missouri State Highway Department, Jefferson City). The purpose of these tests was to

determine the relative efficiency of various curing methods in keeping concrete moist during the curing period. Weighed samples of concrete were placed in enameled pans 12 in. in diameter and 4 in. deep. Two samples were cured by each of the following methods:

(1) Wet burlap for eighteen hours followed by
  (a) Calcium chloride
  (b) Sodium silicate
(2) Curcrete applied immediately after molding.
(3) Hunt Process applied immediately after molding.
(4) Calcium chloride used integrally.

Though these tests have been completed, no report has yet been prepared for publication.

Effect of Different Methods of Curing on Strength of Concrete: (Pennsylvania Department of Highways, Harrisburg). This investigation covered a comparison of the value of wet straw, calcium chloride used integrally, and asphaltic emulsion sprayed on the surface as curing agents. Sections of a state highway were cured by these methods. Specimens were molded for strength determinations at various ages, and cores were taken from the road at the six-month and one-year periods. The strength of beams, cylinders, and cores was about equal. At the six-month period the calicum chloride and straw-cured cores gave a strength of 4100 lb. while those cured with the asphaltic emulsion were slightly lower. The beams and cylinders cured with calcium chloride were slightly higher at 7 and 10 days than either of the others, but at the 14 and 28-day periods the strengths were about equal.

Methods of Curing Concrete: (Iowa State Highway Commission, Ames). This problem was concerned particularly with curing of concrete pavement slabs. The investigation was conducted in four groups, each of which consisted of tests on slabs 10 ft. by 10 ft. by 6 in., made in a manner approximating ideal field conditions, and including from four to fifteen methods of curing. In the construction of the slabs longitudinal divider boards were spaced two and one-half feet apart, thus dividing the slabs into four strips 2½ ft. wide and 11 ft. long. These strips were removed one day before testing at ages of 7, 28, 90 and 360 days. Each strip was sawed transversely into 6 by 6 by 30-in. beams on which flexural tests were made. Compression tests were made on specimens prepared from the beams. The results indicate that the wet burlap moist earth method of curing is the most efficient method available; that other methods afford strength results of varying percentages of the results obtained for this method; that asphaltic coatings such as Curcrete and Hunt Process are 90 and 95 per cent as effective as this method; that calcium chloride on the surface is about the same; and that the original water content of the concrete must be maintained or even augmented for the most efficient curing.

A laboratory study has been undertaken to investigate the latter point, and to determine the effect of different methods of curing on cracking, scaling, and

volume changes of the concrete. The work on volume changes is well under way and the data obtained indicate greater daily volume changes for asphaltic coatings than for the other methods, especially the standard earth and water method.

Comparative Study of Wet Earth, Calcium Chloride, and Sodium Silicate for Curing Concrete Pavements: (U. S. Bureau of Public Roads, Washington, in cooperation with the Maryland State Roads Commission). A description of this project as well as the results of tests made up to one year was published in the September, 1928, issue of Public Roads. Observation of service behavior is made from time to time.

DAMS

Temperature Studies in an Arch Dam: (University of Alabama, University, Ala.). The purpose of this investigation was to determine the maximum temperature occurring in concrete after placing, the temperature at time of closure of the dam, and the annual range in temperature. Thermocouples were set in the concrete at time of construction, and readings have been taken each month for the past year. This investigation will probably be continued for another year.

Investigation of Models of Arch Dams: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Engineering Foundation). This extensive study was undertaken to add to existing knowledge of stress distribution for design purposes, and is a part of the preliminary investigation of the Boulder Canyon Dam. The report of this Committee for 1928 (see page 444, Proceedings, A. C. I.), carried a complete outline of tests to be made on a model of the Stevenson Creek Dam, which have since been completed. Experimental work of similar nature was carried out on a model of the Gibson Dam, with auxiliary tests in flexure, tension, torsion, compression, volume changes, etc., somewhat less elaborate than those carried out on the Stevenson Creek model. It appears from the results obtained to date that models have great possibilities as aids to structural analysis of dams, as have also the auxiliary findings.

Model Tests on Hoover Dam: (U. S. Bureau of Reclamation at the University of Colorado, Boulder, in conjunction with Arch Dam Committee of Engineering Foundation). A model of plaster of Paris diluted with Celite to reduce the modulus of elasticity sufficiently to give measurable deformations is under construction. Young's modulus will be between 100,000 and 300,000 lb. per sq. in. and the model will be to a scale of about 1:200. Very extensive tests on cylinders (2 by 4-in., 3 by 6-in., and 6 by 12-in., on 3 by 3 by 40-in. and 4 by 6 by 38-in. beams) have been made on a great variety of mixtures and materials in finally selecting the materials and proportions to be used in the model. Extensive tests on the rate of drying out have been made, since plaster of Paris is plastic when moist and must therefore be in a dry condition at test. The methods of test will be similar to those used for the Stevenson Creek and Gibson models.

DURABILITY

Durability of Concrete: (Hydro-Electric Power Commission, Toronto, Canada). During the past year most of the work done in this investigation was a field study of the behavior of concrete structures under different conditions of service, with a total inspection of about seventy-five structures. In addition, laboratory studies are being made of different natural waters to determine their corrosive effect on concrete. The monthly variation in composition is being obtained of fifteen different waters ranging in type from a very pure, soft water, originating from swamps to a relatively hard water. The products resulting from the seepage of these various waters through the concrete are being determined, and an attempt is being made to duplicate in the laboratory the action of corrosive water on concrete as observed in the field.

A 3-yr. study of the efficacy of colorless waterproofing treatments for concrete surfaces, which forms a part of this investigation, has been completed. The results indicate that few of these treatments have any permanent waterproofing or protective value when applied to concrete surfaces.

Methods of Rendering Concrete Drain Tile Resistant to Alkali Attack: (Iowa State College, Ames). This project covers long-time observations of concrete samples which were treated with various chemicals and inert materials before immersion in alkali solutions. A report of this investigation will be published during 1930.

Resistance of Concrete to Waters of Acid Reaction: (State of Washington Department of Highways, Olympia). The purpose of this extensive investigation, which was started in 1927 and which will probably not be completed before 1931, is to determine the nature and extent of attack by waters having an acid reaction, and to study means of making concrete resistant to such attack. One series of tests has already been completed, and another is under way on tests of specimens exposed to artificially acidified slowly running water in the laboratory, and to natural waters of acid reaction. It is intended that the second series will cover such points as (1) relationship between volume of concrete and surface exposed, (2) depth of attack from the surface, (3) protection afforded by coatings and admixtures, and (4) comparison between artificially acidified water in the laboratory and a naturally acid stream.

Durability of Concrete: (Kansas State Agricultural College, Manhattan). The purpose of this investigation was to determine the resistance of concrete and concrete aggregates to alternate freezing and thawing and to exposure to alkali water, as well as to determine the effect of porosity on durability. Freezing and thawing tests were made on 300 concrete specimens of various water-cement ratios and mixes, and on 200 aggregate samples.

Field specimens were placed in 4 locations in Kansas streams known to contain alkali. The following are some of the results obtained:

(1) Field specimens of 1:5 or leaner mix show continued disintegration. Specimens of mixes richer than 1:5 are unaffected.

(2) From tests of specimens exposed to freezing and thawing it appears that:

(a) Alternate freezing of saturated concrete at low temperatures and thawing at room temperatures are a valuable means of determining durability of concrete.

(b) Variations in water-cement ratio of plastic, workable concrete have a very marked effect on its durability. A water-cement ratio of 1.0 is near the maximum permissible to obtain desirable durability of concrete under conditions of exposure.

(c) Use of unsound aggregate results in unsound concrete the resistance of the mortar to distintegration being only slightly effective in protecting the aggregate.

Deterioration of Concrete Silos: (Kansas State Agricultural College, Manhattan). The purpose of this investigation was to determine the causes of deterioration of portland cement concrete when exposed to action of ensilage liquors, and to develop methods of overcoming and preventing such deterioration. The tests were undertaken as the direct result of numerous requests received during the past five years for information on means of protecting the interior surfaces of silos from deterioration by the ensilage juices.

In carrying out this investigation, portland cement concrete specimens were prepared with varying mixtures and varying types of surface treatments, and tested for permeability before and after exposure to the silage juices. Synthetic solutions of ensilage liquors were prepared and the specimens were exposed to the action of these solutions under varying conditions of submergence, air exposure, temperature and concentration of solution. The interiors of five silos in the vicinity of Manhattan were treated with ten different protective coatings. The interior of each silo was divided into equal areas (usually 6 or 8), extending the full length of the silo, each area being treated with a different material. These treated silos are being examined periodically to determine the effectiveness of the surface coatings.

Protective Treatments for Concrete against the Action of Salt Waters: (U. S. Bureau of Public Roads, Washington). Mortar specimens treated in various ways are installed in the alkali water of Medicine Lake, South Dakota, and their condition noted from time to time.

FATIGUE

Fatigue Tests on Concrete: (Iowa State Highway Commission, Ames). Studies are under way to determine the effect of repetition of stresses on concrete. Tests were made on 6 by 12-in. and 4½ by 9-in. cylinders to determine effect of repeated loadings on modulus of elasticity and strength in compression. Some work was also done on the number of repetitions at different percentages of the ultimate loading. Apparently a relatively small number of repetitions of 62.5 and 75.0 per cent of the ultimate stress will cause failure.

FIRE RESISTANCE

Fire Resistance of Walls of Concrete Masonry Units: (Research Laboratory of the Portland Cement Association, Chicago). In this investigation the

various factors bearing on the performance of walls of concrete masonry units when subjected to standard fire exposure are being studied. The investigation was started in June, 1928 with the design and construction of suitable equipment for the conduct of standard fire tests and the installation of a block machine and mixer for the manufacture of the units. To date fire tests have been conducted on more than 80 wall panels 5½ by 6 ft. in size representing approximately two thirds of the test work scheduled for the investigation.

The test program is arranged to determine the influence of such factors as grading of aggregate, cement content, character of aggregate, design of masonry units, type of mortar, workmanship in laying up the walls, and application of plaster. In general, the tests are scheduled in groups or series to bring out the effects of each factor over a wide range. As the work progressed it was found desirable to include several miscellaneous tests. Valuable information is being obtained regarding the stability and safety of walls of concrete masonry both before and after severe fire exposure.

JIGGING

Effect of Jigging during Setting Period on Compressive Strength of Concrete: (University of California, Berkeley, Calif.). Tests were made on 1:4 concrete having a ¼-in. slump. For one group, the aggregate had an appreciable excess of fines, and in another group the proportion of fine aggregate was the minimum which would produce a workable mix. After molding, the specimens were subjected to jigging, which varied from one-half to 1½ hours and from 33 to 154 vibrations per minute, in accordance with methods described in the 1927 report of this Committee.

The maximum increase in strength of the jigged over corresponding unjigged specimens was 40 per cent. The average increase for Group 1 was 13 per cent and for Group 2, 19 per cent. The highest rate of jigging gave the best results for each group. For this condition, the average increase in strength for Group 1 was 31 per cent, and for Group 2, 23 per cent.

A second series of tests was conducted using two water-cement ratios, two gradations of aggregates, and two diatomaceous earths in varying proportions. The compressive strength was increased by jigging for all mixes used, and increased with time of jigging up to the maximum of one hour used in this series. Concrete made from an aggregate with an excess of fine material was benefited less than that made from a properly proportioned aggregate.

FLOW UNDER LOAD

Flow of Concrete under Continuous Compressive Stress: (University of California, Berkeley.) This investigation is being conducted in 3 series, the first of which includes tests on 4 by 14-in. cylinders subjected to constant compressive stress at 70 degrees F. as previously described in the 1928 report of this Committee.

The second series of flow tests has been started to determine the effect of type of aggregate and the effect of moisture conditions upon flow. All specimens were loaded at 28 days to 800 lb. per sq. in. The net flows (after cor-

recting for the changes in unloaded control specimens) at age of 90 days were as follows:

Type of Aggregate	*Flow in Millionths of Inches per Inch*
Sandstone	710
Basalt	680
Gravel	565
Granite	465
Quartz	387
Limestone	313

The third series of tests to determine the relative longitudinal and lateral flow of plain and reinforced concrete cylinders is now in progress. The cylinders are 10 in. in diameter and 20 in. long, two of them being reinforced with six ½-in. square bars enclosed within an 8-in. spiral. One reinforced cylinder and 1 plain cylinder were loaded to produce a stress of 800 lb. per sq. in. in the concrete. Test observations up to the age of 90 days show that the deformation due to load in the reinforced specimen is approximately ⅔ the deformation of the plain specimen at all ages. For the unloaded specimens the shrinkage deformation of the reinforced specimen is approximately ¾ that of the plain.

Flow of Concrete under Continuous Compressive, Flexural, and Tensile Stress: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Arch Dam Committee of Engineering Foundation. See Proc. A. C. I., 1929, p. 444 for outline). Data have been obtained for loadings at various ages under air and water curing and the results are being worked up for publication. In general the findings accord with the published results of Davis and others in so far as the tests are parallel.

Plastic Flow of Concrete: (Ohio State University, Columbus). A correlation is contemplated of all work carried out on this subject to date for the purpose of publishing it, together with methods of application to engineering design computations.

MIXERS AND MIXING

Investigation of Concrete Paver Operation: (Minnesota Highway Department, St. Paul). The purpose of this experiment was to check the water supply equipment in use and the amount of concrete remaining in mixer drums from one batch to the next, and to make a time study of the mixer cycle on Minnesota paving jobs. Stop watch studies of the mixer cycle, water measuring tanks were calibrated, and the water supply equipment was inspected. The latter was usually found to be adequate, though the valves on water measuring tanks were inefficient in several cases. There was considerable variation in different mixers in time of charging, discharging, and entrance of water into the drum, which resulted in a variation in the time interval necessary for mixing and for complete cycle.

MODULUS OF ELASTICITY AND POISSON'S RATIO

Effect of Moisture on Strength and Modulus of Elasticity of Concrete: (University of California, Berkeley). Tests were made at ages of 35 and 142 days

on 6 and 12-in. cylinders of three water-cement ratios. Half the specimens were left in water until the time of test, and the remainder removed from the water and stored in dry air at 130 degrees F. for 7 days prior to test. All were tested at 70 degrees F. The results are given below:

Mix	*Strength, lb. per sq. in.*				*Secant Modulus at 800 lb. per sq. in.*			
	35 Days		*142 Days*		*35 Days*		*142 Days*	
	Dry	*Wet*	*Dry*	*Wet*	*Dry*	*Wet*	*Dry*	*Wet*
1:1⅔:3	3740	3150	5940	5400	2,770,000	3,270,000	4,060,000	4,640,000
1:2½:3	3120	2470	4870	3840	2,510,000	2,940,000	2,960,000	3,520,000
1:3¾:4½	1590	1400	2520	1695	2,020,000	2,740,000	2,100,000	2,800,000

It is evident that the effect of moisture is to lower the strength and to increase the modulus of elasticity, the strength range being relatively greater for rich mixes than for lean ones, and the modulus of elasticity range being relatively greater for lean mixes than for rich ones.

Modulus of Elasticity of Concrete (State College of Washington, Pullman). Data have been collected from the results of routine testing of concrete cylinders in the field as well as in the laboratory to determine whether concrete can be considered elastic, and if so, to determine the relation between the proportional limit and ultimate strength, as well as to determine the modulus of elasticity. The results of these tests were published in Bulletin 30 of the Engineering Experiment Station, in April, 1930.

Modulus of Elasticity and Poisson's Ratio of Concrete: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Arch Dam Committee of Engineering Foundation). In connection with all compression tests now made in this laboratory both lateral and longitudinal strains are measured continuously to failure. A semi-autographic apparatus is used in connection with dial extensometers and compressometers. Hundreds of tests have been conducted in this manner and quantities of data have been obtained from 2 by 4-in., 3 by 6-in. and 6 by 12-in. compressive specimens for many different mixtures, aggregates, ages, and curing conditions. Some of these data are in preparation for publication.

Modulus of Elasticity and Poisson's Ratio (National Bureau of Standards, Washington). Tests have just been started on 24 by 48-in. cylinders made at the site of the Calderwood Dam in Tennessee from concrete containing crushed stone fragments ranging in size from five to six inches in diameter, to determine the deformations under load. Longitudinal deformations are being measured during the loading over 32-in. gage lengths along 6 lines equally spaced around the periphery, and also in 2-in. gage lengths at five different elevations. In addition, transverse deformations are being measured in five diameters at different elevations. It is hoped that the data obtained will be sufficient for a satisfactory determination of the modulus of elasticity and Poisson's ratio of the concrete.

Elastic Properties of Concrete: (Kansas State Agricultural College, Manhattan). This investigation has just been undertaken to determine the

modulus of elasticity and Poisson's ratio for concrete, particularly for that made from sand and gravel. At present the extent of the investigation is the trial of several different types of measuring instruments and the development of a new type.

Modulus of Elasticity in Tension, Torsion, and Flexure: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation and Arch Dam Committee of Engineering Foundation). Many tests have been made on 3 by 12-in. tensile and torsion specimens, and on 3 by 3 by 40-in., and 4 by 6 by 38-in. beams for different ages and curings for the mixtures of the Stevenson Creek and Gibson models. Loads and elongations were taken for tensile specimens; loads and detrusions for torsional specimens; and loads and deflections for flexural specimens. The increment method was used since semi-autographic facilities for continuous testing are not available for these tests. Results are in preparation for publication.

PAVEMENTS

Stresses in Concrete Pavement Slabs under Concentrated Static Loads: (Iowa State College, Ames). This project was started in the spring of 1929 and complete data have been assembled on tests of one small slab, a report on which will be available at an early date.

Correlation of Pavement Condition and Soil Survey: (Michigan State Highway Department, Lansing). Tests were started in 1925 to determine and evaluate those factors which influence the condition of pavements. Three thousand miles of pavement have been studied, including field sketches of pavement condition summarized slab by slab, and correlation of the condition with slab characteristics, such as design, age, traffic, material, curing, reinforcement, character of subgrade, old road beds, and drainage. No definite conclusions are available at the present time.

Study of Methods Commonly Used for Removing Ice from Concrete Pavement Surfaces: Michigan State Highway Department, Lansing). This investigation is being made to determine the most economical and efficient material for removing ice with no detrimental effect on the concrete. Laboratory beams were made, frozen a number of times, and the surface film of ice removed with an application of sodium chloride, calcium chloride, or hot sand. Subsequent observations will be made of the surface condition when exposed to drying atmosphere.

High Early Strength Concrete in Pavements: (Pennsylvania Department of Highways, Harrisburg). A service test road was constructed to determine the relative efficiency of high early strength concrete obtained by (a) high early strength cement, and (b) a special mix. Sections of the road were built using high early strength cement in a 1:2:3¾ mix, standard portland cement in the same proportions and standard portland cement in a 1:1½:2½ mix. The project included four traffic lanes, two of which contained high early strength cement, and the other two the standard portland cement used in the 1:2:3¾ and 1:1½:2½ mixes. After approximately one year of traffic the road sections are all in excellent condition. The special mix and the mix using high

early strength cement were opened at one and two-day periods, depending on the beam strength of the concrete.

Effect of Variable Subgrade Conditions, Various Construction Practices, and the Use of Joints of Various Types on the Service Behavior of Concrete Roads: (U. S. Bureau of Public Roads, Washington, in cooperation with the Virginia State Highway Commission). A description of the various test sections of the Virginia Demonstration Road was published in the June, 1928, issue of Public Roads. Observations of the behavior test sections are being made from time to time.

PERMEABILITY

Permeability Tests of Concrete: (Kansas State Agricultural College, Manhattan). Permeability tests were made on 440 mortar specimens of various mixes and water-cement ratios at pressures varying from 10 to 100 lb. per sq. in. In these tests 3 by ¾ in. cylinders were cured in the moist closet for 24 hours and stored until test in stone jars containing distilled water. The water coming through the specimens was timed for rate of flow, and then titrated with N/20 HCl, using phenophthalein and methyl orange as indicators to determine the amount of lime which was dissolved. In another group of tests 1½-in. discs were sawed from 6 by 24-in. concrete cylinders made from portland cement and Quikard cement in the following mixes: Cement, sand from the Kansas River with a fineness modulus of 2.94, and crushed sandstone in the proportions of 1:1¾:3½, cement and total aggregate in the proportions of 1:3½, the water-cement ratio in each case being 0.7.

The results thus far show that a 1:2 mix was watertight at any pressure up to and including 100 lb. per sq. in. if enough water was used to make a good workable mix, but not so much that the mix is sloppy. That is, a water-ratio of 0.6 to 1.0 may be used, but 0.65 to 0.8 would be better. A 1:3 mix was watertight between the water-ratios of 0.6 and 1.0 and a 1:4 mix between the water-ratios of 0.7 and 1.0 at all pressures up to 100 lb. per sq. in. A 1:5 mix with a water-cement ratio of 0.8 was watertight up to 60 lb. pressure, but water flowed through when more pressure was applied, or if a higher or lower water-cement ratio was used.

Permeability of Concrete: (University of California, Berkeley). An investigation has been outlined to determine the effect of water-cement ratio on permeability of 2 by 6-in. discs.

Permeability Studies: (Portland Cement Association Research Laboratory, Chicago). Studies were inaugurated in the spring of 1928 to determine the influence of various factors on the watertightness of concrete. Apparatus was developed which permitted the making of simultaneous tests on a large number of specimens. The apparatus is so constructed as to permit measurement of the water passing through the specimens which consist of 6-in. diameter discs of varying thickness. The specimens may either be cast in special molds or cut from standard 6 by 12-in. cylinders.

Thus far tests have been made to determine the effect of the following factors on the permeability of concrete: Quality and quantity of cement paste, quantity and type of cement, duration of moist and air-curing, age at test, duration of the test period, thickness of specimen, amount of pressure applied, type of unit, method of molding, direction of application of pressure, uniformity of results, and influence of powdered and liquid admixtures. Results of some of the early tests are reported in the December, 1929, JOURNAL of the Institute.

Oilproofing Concrete (Detroit Edison Co., Detroit, Michigan). The purpose of this investigation was to develop a concrete, or a surface treatment for concrete, which would prevent the absorption of oil from transformers. Seven commercial surface coatings, 3 special cements, 2 integral compounds, and 10 concrete toppings of various mixes, aggregates, and methods of application were investigated. The concrete was molded into saucers 22 by 22 by 6-in. with a 2-in. depression in the top to hold the oil. These specimens were cured for 7 days, dried out for 14 days, treated with various coatings, and filled with light oil of the type used in transformers, the oil being left in the depression for various periods between 9 and 28 days. The specimens were then broken and a careful study made of the cross sections. The results obtained show that concrete can be made or surface treated in such a manner that it will shed oil without any absorption.

Waterproofing Concrete: (Iowa State Highway Commission, Ames). In this investigation tests were made to determine the efficacy of various methods of waterproofing concrete by surface treatments. A number of specimens were coated or painted and immersed in water, after which they were removed and weighed at fixed intervals. From the results obtained it appears that several relatively cheap asphaltic coatings would be quite satisfactory as waterproofers for concrete.

PIPE

Study of Loads on Concrete Pipe in Embankments: (Iowa State College, Ames). This study was continued with the collection of special data on the correlation of laboratory strength of pipe with field strength, and additional studies were made to determine the settlement, changes in diameter, and other phenomena necessary to correct interpretation of the loads caused by embankments.

Strength of Concrete-Encased Clay Sewer Pipe: (Iowa State College, Ames). The results of this investigation were published in the autumn of 1929 in Bulletin 93 of the Iowa Engineering Experiment Station. In addition to the results of strength tests of encased clay pipe, the bulletin includes data on the effect of temperature on strength of concrete.

PRODUCTS

Physical Properties of Cast Stone: (National Bureau of Standards, Washington). This study was undertaken with a view to establishing a federal specification for cast stone. Compression tests were made on 2 by 2-in. cylinders and the modulus of rupture was determined on 1 by 1-in. prisms using a 6-in. span.

In addition, rate of absorption tests were made, and weathering effect was determined by freezing at a temperature of 10 degrees F.

STRENGTH, COMPRESSIVE

Strength of Cement Mortar and Concrete: (Iowa State Highway Commission, Ames). The purpose of this study was to determine the relationship between the strength of cement mortar and the strength of concrete. Particular attention was given to comparison of values obtained in tensile tests of standard mortar briquets and compression tests of field specimens made during the construction of paving projects. The mortar strength tests were made as a part of the routine tests conducted for the acceptance of cement shipped to each paving project. Correlation of these tests with strength tests on field specimens was found to be practically impossible because of the number of variables encountered and the complexity of their relationships. Though work has been temporarily discontinued, these studies will be resumed when the effect of the variables can be more accurately evaluated.

Effect of Adding One-Half to Three-Quarters Inch Gravel and Also One-Half to Three-Quarters Crushed Quartzite Particles to a Neat Cement Paste: (University of Colorado, Boulder). These were miniature "plum or cobble" concretes (specimens about 3¼ inches in diameter). They were cast in watertight molds and only such an amount of stone as could be accommodated easily was added. The "cobble" specimens had compressive strengths about half those of the same paste without any "cobbles." The water-cement ratio was 42 per cent by weight. The weakening was probably due in part to lack of homogeneity (stone bearing on stone, splitting and wedging action) but may also have been traceable to bond failures between stone particles and paste.

STRENGTH, FLEXURAL

Relationship between Transverse and Compressive Strength of Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). A compilation was made of all available results obtained since 1925 in an effort to determine whether transverse or compressive strength results should be considered the fairer criterion of the quality of a concrete. Transverse tests were made using third point loading, and compression tests were made either on 6 by 12-in. cylinders or on pieces cut from broken beams. Below is a brief tabulation of the more important results:

Age in Days	*Strength, lb. per sq. in.*		*Factor*	*Number of Tests*
	Transverse	*Compressive*		
28	1010	3833	3.82	12
21	934	3737	3.99	12
14	936	3596	3.83	12
7	752	2833	3.76	4
28	616	3761	6.08	30

In addition, 23 tests at 28 days showed from 440 to 1141 lb. transverse strength and from 2250 to 5386 lb. in compression with a factor of from 3.84

to 7.03. It is evident that the aggregate is a factor affecting the transverse and compressive strength of concrete, and the results indicate that the compression test is the more likely to reflect the true value of the material. It seems feasible to state that concrete giving satisfactory compressive strength will also give satisfactory strength in flexure, and the resultant concrete will function properly in a pavement.

Comparison of Compressive Strength and Modulus of Rupture: (University of California, Berkeley). For the mixes tested, giving compressive strengths of 2000 to 4000 lb. per sq. in. at 28 days, the modulus of rupture was found to be approximately

$$(S_C - 2000)\ .11 + 450$$

where S_C is the compressive strength of the concrete.
The actual modulus may vary from the computed value by as much as 90 lb. per sq. in.

Masonry Flexure Tests: (University of California, Berkeley). Tests are in progress to determine the flexural strength of brick and tile masonry beams using a variety of cement-lime mortars, the beams varying in size from 8 by 4 by 24-in. to 24 by 12 by 54-in. Nearly 500 specimens are to be tested, including a group of adhesion and bond specimens.

Flexural Tests of Plain Concrete Beams: (New Hampshire University, Durham). This study was undertaken to determine the relation in modulus of rupture between the standard Olsen Laboratory Machine and certain field testing machines, thereby determining the exact stress in beams broken in the field. One hundred sixteen beams were made from water-cement ratio concrete designed for strengths of 2000, 2500, 3000, 4000, and 5000 lb. per sq. in. Such variable factors as curing, time of mixing, size of aggregate, etc., were carefully controlled to insure uniformity in concrete. Companion specimens for each batch were tested on each type of machine, the beams tested in laboratory on the Olsen machine being loaded at the third points of the span.

Standardization of Method for Determining Flexural Strength of Plain Concrete: (American Association of State Highway Officials). The purpose of this investigation was to design and adopt a standard apparatus and method of molding specimens for the determination of the flexural strength of plain concrete. Special apparatus has been designed and built, which is to be used in conjunction with a power testing machine. It is so designed that all forces applied to the specimen are coplanar and perpendicular to its axis, and the deformation due to applied load is entirely unrestrained.

Standardization of Methods of Making Flexural Tests of Concrete: (U. S. Bureau of Public Roads, Washington). These tests are being made in cooperation with A. S. T. M. Committee C-9 on "Concrete and Concrete Aggregates."

TEMPERATURE EFFECTS

Effect of Temperature at Time of Test on Strength of Concrete: (University of California, Berkeley). Tests were conducted on 6 by 12-in. concrete cylin-

ders of 1:2:4 and 1:3:6 mixes at ages of 28 and 135 days. All specimens were wet-cured, but a few days prior to test various groups were subjected to temperatures ranging from 25 degrees F. to 130 degrees F.; some were dried while others were kept wet. In general, the test results show a progressive decrease in strength with an increase in temperature at time of test. The maximum effect developed in the 1:2:4 wet specimens tested at 135 days, the strength ranging from 140 per cent to 85 per cent of the strength at 70 degrees F. The leaner mixes were not affected so markedly as the 1:2:4 mixes, nor the dry specimens so much as the wet.

Effect of High Curing Temperature on Strength of Concrete: (University of California, Berkeley). Tests were made on 2 by 4-in. mortar cylinders of 1:2½ and 1:5 mixes by volume. The curing temperatures were 70, 100, 140 and 180 degrees F.; the time of curing 18, 24, and 36 hours for each temperature, a fog being maintained during this period by means of an atomizer. Some of the specimens were stored dry at 50 per cent relative humidity, and some were stored wet until time of test at various ages. High temperatures increased the early strength of cement mortars, the maximum effect being produced by a temperature of 140 degrees F. maintained for from 24 to 36 hours. In general, at ages of 7 days or more, a curing temperature of 180 degrees F. produced lower strengths than a temperature of 100 degrees F. After the age of 7 days, specimens cured at normal temperatures and stored in water exhibited higher compressive strengths then did similar specimens cured at high temperatures and stored in water.

Further tests are in progress on 3-ft. cubes of concrete insulated on all sides, which indicate to date that mass concrete develops a temperature of about 155 degrees F. within 20 hours when using 9 sacks of cement per cubic yard. The 1-day strength of the mass concrete is much greater than the strength of corresponding 6 by 12-in. cylinders cured at 70 degrees F.

Thermal Expansion of Concrete: (University of California, Berkeley). Tests are in progress to determine variations in coefficient of thermal expansion in which variables are water-cement ratio, richness of mix, type of aggregate, curing condition, and age of specimen. Tests on 3 by 3 by 40-in. bars for temperatures between 15 and 130 degrees F. indicate the following results:

(1) The coefficient of thermal expansion varies with the richness of the mix, the value for a 1:3 mix being about 6 per cent higher than for a 1:6 mix of similar materials.

(2) The coefficient is about 5 per cent higher for water-soaked specimens than for air-dry concrete.

(3) There is no appreciable variation in the coefficient for ordinary ranges of temperature.

(4) The variation in the coefficient for mixes of various consistencies is negligible.

(5) The type of aggregate has a marked influence on the coefficient, the actual values for 1:4½ mixes of aggregates of identical gradations being as follows:

Quartz	0.0000066	per 1 degree F.
Sandstone	0.0000065	" "
Gravel	0.0000060	" "
Granite	0.0000053	" "
Basalt	0.0000048	" "
Limestone	0.0000038	" "

(6) The coefficient of expansion for air-dry concrete cured normally for 2 months and then subjected to 100 alternations of temperature between 15 and 130 degrees F. was decreased about 14 per cent from the value obtained for concrete not subjected to alternations of high and low temperatures, but stored at a constant temperature of 70 degrees F.

Variations in Temperature in Plain Concrete Slabs as Affected by Type of Curing: (Texas A. and M. College, College Station). Investigations have been started to determine variations in temperature within concrete slabs. Tests are to be made in the field, and temperature records taken with resistance wires. Laboratory tests will be made to parallel the field investigation, using the same materials in 6 by 6 by 32-in. beams and 6 by 12-in. cylinders.

TESTS AND TEST METHODS

Effect of Removing the Water that Gathers at the Top of Mortar or Concrete Soon After Placing: (University of Colorado, Boulder). Comparison specimens were made in water-tight molds. As the water gathered it was carefully removed by blotters from some of these. The amount removed was determined by weighing the specimens at intervals. Except in the case of very rich mixtures and a great excess of mixing water, the water that gathered at the tops of the specimens that were not blotted returned within the mass after a few hours. All specimens were stored in water after one or two days. Those which had been blotted absorbed more water during the early storage period but the strengths of the two classes were practically identical. The shrinkages were equal and it is evident that the removal of the water did not add to the amount of settlement (decrease the voids) and that the effective water-cement ratio was not changed from what it was for the specimens from which no water was removed. These results would indicate that it is incorrect to use a corrected water-cement as a measure of leakage effect unless added compaction results from the loss or removal of the water. It has been demonstrated that any method of water removal that does cause the particles to move closer together will lower the water-cement ratio and increase the strength and density.

Variations in Strength of Concrete Cylinders Due to Rodding: (Kentucky State Highway Department and University of Kentucky, Lexington). The purpose of these tests was to determine variations in strength of cylinders not rodded in accordance with standard methods. The only variables permitted in the investigation were the methods of rodding and placing, which consisted of: (a) use of standard rod with 4-in. layers of concrete varying the number of roddings; (b) use of standard rod with 25 roddings varying the thickness of the layers; (c) 4-in. layers rodded 25 times with a variation in size of rod from

¼-in. round to a 2 by 4-in. wooden rod; and (d) placing without rodding or tamping, but pushing down with trowel. It was found that (1) standard rodding produced satisfactory results both as to strength and density, while (2) concrete thoroughly rodded with a ⅝-in. or larger rod, total roddings being as many as 75 in some cases, gave satisfactory strength, but was less dense than concrete rodded in accordance with standard methods. It may be concluded that if reasonable care is used in tamping concrete into the mold and sufficient tamping is done, use of a standard rod is not important.

Relationship between End Condition and Strength of Concrete Cylinders: (Kentucky State Highway Department and University of Kentucky, Lexington). Studies were made to determine the value of true end condition and the most effective way of obtaining it. Tests were made at 28 days on specimens essentially alike except for end conditions, which varied from true to $\frac{3}{16}$-in. concave and $\frac{3}{16}$-in. convex, in addition to one set which had 14 per cent end area chipped off. Cement mortar and sand bearings were used. The following are the outstanding conclusions drawn from the results: (1) the end condition of a cylinder should be carefully noted upon its arrival at the Laboratory, and if convex should be rubbed to a plane surface or capped with a suitable cap. In no case of this type should sand bearing be used; (2) if the end is concave, it can be rectified by rubbing down, or a thin sand bearing of confined sand, not to exceed ⅛ in. can be used; (3) if the specimen is slightly irregular, as in the case of finishing with a trowel, it should preferably be rubbed down to a smooth surface, though a thin sand bearing can be used; (4) reduction of 14 per cent in end area has no effect on strength obtained; (5) if the true value of a concrete is to be obtained from the cylinder it must be made with perfect bearings on each end. Fair to good results may be obtained with the sand bearing or added cap, but neither of these methods are reliable, and it is important that the ends of the cylinders be correctly finished. This laboratory has recently built and installed a cup grinding machine for finishing cylinders, but results from the use of this machine are not yet available.

Effect of Size of Cylinder on Strength of Concrete: (University of California, Berkeley). Cylinders varying from 2 to 12 inches in diameter and two diameters in length were made from an identical mix, using ½-in. maximum size of aggregate. One group of specimens was stored in air, while a second group was coated with Inertol and stored in a continual fog. Variations in moisture content of test specimens of various sizes were the cause of considerable variations in strength. The strength was not materially influenced by size of test cylinder if all other conditions, including the percentage of moisture in the specimen at time of test, were maintained constant, and if the maximum size of aggregate was small with respect to the diameter of the cylinder. With ½-in. aggregate the strength was not influenced, provided the diameter of the cylinder was not less than 3 inches (6 times the maximum size of aggregate).

Effect of Size of Cylinder on Strength of Concrete and Mortars: (University of Colorado, Boulder). Several hundred of comparative tests have been made

on 2 by 4-in., 3 by 6-in., and 6 by 12-in. cylinders of mortars and concretes in which the aggregates did not exceed ¾ in. (¼ diameter of 2-in. cylinder).

For similar curing and test conditions, the variations between specimens of different sizes is no greater than between companion specimens of the same size. Lateral extensometers and longitudinal compressometers and semi-autographic recording devices have been developed and used in the majority of these tests. The stress-strain curves, both lateral and longitudinal, are similar throughout. Recognition of this as fact opens up a fruitful field for research by smaller laboratories in the realm of one and two-man projects in which studies can be made on results from the smaller sizes of specimens. These tests included varied curings and test conditions and for these cases the agreement is much less perfect because of the differences in the rates of drying out, etc. for the different sizes of specimens. A number of tests were made on 2 by 4-in. specimens in which the ¾-in. maximum size of coarse aggregate particle was exceeded, particles up to nearly 2-in. being included in some cases. For such specimens the strengths were much more erratic but the maximum strengths attained were about the same as those from the same concrete in larger specimens. The minimum strengths were much lower, however, depending upon how much such large particles lay in the specimen. The least strengths were obtained when large particles lay on a bias. Failure then occurred by sliding along the inclined bedding plane. Many of these results are in process of preparation for publication.

Study of Methods for Determining the Height of Cores Drilled from Concrete Structures: (U. S. Bureau of Public Roads, Washington). These tests are being made in cooperation with A. S. T. M. Committee C-9 on "Concrete and Concrete Aggregates" to develop a standard way of determining height of cored cylinders.

VOLUME CHANGES

Volume Changes in Concrete Due to Changes in Moisture Conditions: (University of California, Berkeley). This is a continuing investigation, the earlier results of which are presented in the 1928 report. A series of specimens was made using various types of aggregate but for which all other factors such as gradation, cement factor, etc. were constant. Tests show that volume changes are approximately the same for concrete having limestone, quartz and granite as aggregates. Gravel and sandstone concretes exhibit volume changes about twice as great as for the aggregates just mentioned.

A second series of tests of gravel concrete, using 1:4 and 1:6 mixes and water-cement ratios of 0.98, 1.07, and 1.16 for each mix show:

(1) Contraction in air is greater for mixes of high water cement ratio than for those of low water cement ratio, the effect being most pronounced for the rich mixes,

(2) In general the expansion in water is greater for the mixes of low-water cement ratio, but the difference is not marked.

Volume Changes in Concrete Due to Changes in Moisture Conditions: (University of Colorado, Boulder, in cooperation with U. S. Bureau of Reclamation

and Arch Dam Committee of Engineering Foundation). These tests consisted in length measurements taken at intervals on 3 by 3 by 40-in. beams (similar to those used by Davis) and were obtained for the mixtures of both the Stevenson Creek and Gibson models for various combinations of air and water curing. In general the results accord with previously reported findings by Davis and others.

Volume Changes in Concrete: (University of Michigan, Ann Arbor). This investigation was started in 1902 to determine the changes of volume occurring in neat cement and rich mortar bars at various ages under different conditions. Results obtained at various periods have been published in current engineering literature.

Volume Changes of Neat Cements: (Lehigh Portland Cement Co., Allentown, Pa.). This is a continuation of a project reported in 1928 in which data are continually being obtained on the behavior of test bars under various conditions of exposure. Numerous interesting relations have been disclosed or indicated between behavior of bars under normal and accelerated tests and the constitution of the cements used. In this connection, it may be mentioned that a small conditioning room maintained constantly at a temperature of 70 degrees F. and 50 per cent relative humidity affords a means of comparing shrinkage of bars under definite storage conditions. The indications thus far are that the majority of "normal" cements are much alike in their shrinkage characteristics, but enough exceptions are found to warrant extensive studies into the causes and possible control.

Volume Change of Concrete: (Kansas State Agricultural College, Manhattan). The purpose of this investigation was to determine the reason for the relatively greater number of shrinkage cracks which appear in sand or sand-gravel concretes as compared with those in concrete in which both fine and coarse aggregates are used. Readings are being taken at definite periods on 151 specimens (4 by 6 by 24-in. beams), 86 of which are over one year old, and only 30 of which are less than 100 days old, to determine the effect of the following factors on volume changes:

(1) Type of coarse aggregate,
(2) Variation of material used as total aggregate
(3) Type of sand in mortar mix,
(4) Type of sand in concrete mix,
(5) Quantity and type of cement,
(6) Variation in water-cement ratio,
(7) Use of various admixtures.

Data from this investigation were published in a paper by Scholer and Dawley on "Volume Change of Concrete," in the Proceedings of the American Society for Testing Materials for 1930.

Study of Volume Changes: (Research Laboratory, Portland Cement Association, Chicago). Various factors affecting volume change of concrete are being investigated. These include effect of regaging and remixing, size and shape

of specimen, absorptive form, curing, coatings, admixtures, fineness, type and storage of cement, cement content, paste ratio, water-cement ratio, type and kind of aggregates, ratio of fine to coarse aggregate, moisture loss and vibration during mixing. The studies to date have brought out the inter-relation of some of the principal factors involved in these changes and a method has been evolved for estimating the relative shrinkage of various concrete mixtures.

WEAR

Relationship between Cement Factor and Wear and Strength of Concrete: (Kentucky State Highway Department and University of Kentucky, Lexington). The purpose of this investigation was to establish an economical cement factor as a basis of control for pavement concrete. A concrete mix was used in which the fine aggregate was 1.2 times the voids in the coarse aggregate, with a cement factor varying from 1.5 to 1.7 bbl. per cu yd. Comparisons were made of different aggregates in beams, cylinders, and 9-in. balls. Below is a summary of the tests made using stone and gravel:

Cement bbl. per cu. yd.	*Coarse Aggregate*	*Water-Cement Ratio*	*Wear Concrete Balls* %	*28-Day Strength*		*Strength Comp. Specimen at 42 Days*	
				Compressive	*Transverse*	*Cylinders*	*Cores*
1.5	Stone	0.74	19.4	3606	860	5061	4888
	Gravel	0.70	19.4	3206	643	3732	3300
1.6	Stone	0.70	17.5	3273	817	4465	4690
	Gravel	0.63	15.9	3899	712	3856	4300
1.7	Stone	0.68	15.0	4361	912	4521	4725
	Gravel	0.60	11.7	3585	692	3990	3990

The above data indicate that 1.5 bbl. of cement per cu. yd. of concrete with 5¾ gal. of water using aggregate that passed a standard specification will give sufficient strength both in tension and compression to insure good concrete pavement.

There seems to be no good reason why concrete made with crushed stone should require from 0.1 to 0.2 bbl. more cement than that made with gravel as is the case with the 1:2:3.5 mix commonly used.

Workability of Concrete: (Ohio State University, Columbus). As a means of standardizing concrete from central mixing plants, tests are being made to develop a satisfactory workability meter, after which a thorough study will be made in an effort to determine why some mixtures of the same proportions are harsh and others are smooth.

Workability of Portland Cement Mortars: (National Bureau of Standards, Washington). This is a continuation of the study described in the paper on "Cement as a Factor in the Workability of Concrete," which appeared in the 1928 Proceedings of the A. C. I. in which the effect of various portland cements on the workability of mortars is described, as well as a discussion of a means of measuring this workability. Further tests have been made for measurement

of workability as described in the foregoing paper, but at present full attention is being given a modified McMichaels viscosimeter, using various mixes with various water content, as well as various gradings of sand.

4—Reinforced Concrete

BOND

Bond Resistance of Reinforced Concrete Beams under Continued Load: (T. & A. M. Dept., University of Illinois, Urbana). A group of 24 beams were loaded continuously over a period of several months by means of jacks and heavy, calibrated spiral springs to determine the effect of continued loading of various intensities on bond-slip relations. Slips and strains were measured at various points in the beams at regular intervals of time. There was a total of 40 beams in all, together with auxiliary compression and pull-out specimens.

BRIDGES

Study of Behavior of Reinforced Concrete Arches During and Immediately Following Construction: (State Highway Board of Georgia, East Point). This investigation was conducted on an arch bridge near Eatonton, Georgia. Measurements were taken to determine the initial behavior of an open-spandrel reinforced concrete arch rib. Readings were first taken during placing of the concrete in the arch ribs and continued throughout the construction of the bridge and for a month thereafter, a total period of about 4 months. The work included measurement of strain in steel and concrete, beginning with zero strain while the concrete was being placed; determination of the compressive strength, modulus of rupture, and modulus of elasticity of the concrete used in the arch rib at various ages; measurement of the vertical movement of the arch rib at the crown, beginning just after the last section of the rib had been completed; measurement of the movement of abutments; measurement of temperatures at various points in the arch rib; and measurements of movements in expansion joints in the decks. Full details of this investigation were reported in the Proceedings of the American Society of Civil Engineers, page 2279, November, 1929.

Tests on Arlington Memorial Bridge: (National Bureau of Standards, Washington). Tests on one of the arches of the Arlington Memorial Bridge are being made by the National Bureau of Standards in conjunction with the Arlington Memorial Bridge Commission to obtain data on the structural behavior of the arch during and after its construction. Arch span No. 7 selected for testing purposes is 177 ft. 4 in. in clear span, and has a rise from springing line to crown intrados of 26 ft. 8 in. Construction of the arch was begun in October, 1928, and it is now completed, with the exception of the roadway and sidewalk paving surfaces.

The temperature of the concrete in the arch barrel has been measured at intervals since the concrete was placed. Other measurements include the deflections and strains of the arch barrel caused by different temperature and load conditions. The concrete temperature has been measured with resistance thermometers and thermocouples, the thermometers being connected to a

potentiometer recorder for sixteen stations. Deflection observations are obtained by means of clinometers which are used on a continuous line of 5-ft. stations extending over the back of the arch and a portion of each supporting pier.

Strain measurements are made at selected points, strain gages being used on the extrados, and electric telemeters buried in the concrete are used at the intrados. The measurements will be continued until data have been obtained on the effects of temperature changes on the deformations of the arch.

Stress Reduction in Concrete Arches by Means of Hydraulic Jacks During Construction for Adjusting the Axis to its Most Favorable Position: (U. S. Bureau of Public Roads, Washington, in cooperation with the Oregon State Highway Commission). Studies are being made in accordance with Freyssinet's method of reducing stress in concrete arches by means of hydraulic jacks.

Investigation of Reinforced Concrete Columns: (American Concrete Institute Committee 105). An extensive investigation on reinforced concrete columns is being carried out at Lehigh University and at the University of Illinois by Committee 105 of the American Concrete Institute. The purpose of this study is to establish the fundamental relationships between load, elastic properties and ultimate strength of reinforced concrete columns as influenced by variations in size, quality of concrete, curing of concrete, amount, arrangement and quality of reinforcement, fireproofing, rate and method of loading and the effect of continuous loading over long periods of time.

The original program includes a total of 531 reinforced specimens and 64 unreinforced specimens. To this number an additional series of 24 specimens has recently been added.

The size of specimens ranges from 8 in. in diameter by 5 ft. long to 32 in. in diameter by 17 ft. 6 in. long. Most of the specimens, however, are of the smaller size. Concrete strengths vary from 2000 to 8000 pounds. per square inch. Sixty are being held under design loadings over a period of one year during which time periodical strain gage measurements are being made to determine the amount and rate of time yield. Measurements are made on companion specimens not under load at corresponding intervals to determine shrinkage effects.

In 4 out of a total of 6 series comprising the original program the work of the two laboratories is intended to be identical. The larger columns are all being tested at University of Illinois where a 3,000,000 pound hydraulic testing machine has recently been installed. Some of the specimens will utilize this machine to practically its full capacity.

Effect of Stressing Steel Encased in Concrete Above Its Yield Point: (University of Colorado, Boulder). Plain and spirally reinforced 6 by 12-in. concrete cylinders were cast each with a single ½-in. square structural steel bar along its axis. The bars had milled ends and projected about ⅛-in. from each end of the cylinders. This required that a bar be stressed to the yield point of the steel before any load bore directly on the concrete. In some specimens the bars were greased and in others not. There were also controls of plain concrete, and spirally reinforced specimens with no bar. Still other

specimens had the bar ground flush and in others the bar did not extend quite to the ends of the specimens. Both lateral and longitudinal strains were measured. While the findings are not all decisive, two facts were demonstrated in so far as the conditions of these tests are concerned.

(1) The plain concrete specimen with bar projecting, but without spiral, split about the time that the yield point of the steel was reached.

(2) The spirally reinforced cylinders with projecting bars did not split. The bar shortened and became flush with the specimen within its yield point range and without visible damage to the concrete. Finally the bars buckled within the specimen after the maximum load was passed and the concrete badly punished.

These tests were run as a reconnaissance of what might be expected in a concrete column if the yield point of the steel is exceeded due to shrinkage, flow, or other cause. The tentative conclusion is that the result should not be serious in a well-spiralled column but it is to be expected that splitting would occur at the yield point, if not earlier in a tied column or a block of plain concrete, in which similar steel was imbedded.

REINFORCING STEEL

Design of New Type of Steel Reinforcing for Large Concrete Conduits: (Iowa State College, Ames). This investigation has been under way for some time and a patent has been secured on a new steel reinforcing. Tests were made at the Indianapolis plant of the Independent Pipe Company on several sections of concrete pipe reinforced with this special steel. The results of the tests will be available soon.

Investigation of Hooks as Anchorage in Reinforced Concrete: (Washington University, St. Louis, Mo.). The purpose of this investigation is (1) to determine the effect of varying strength of concrete, (2) effect of radius of hook bend, (3) effect of kind of bar (plain or deformed) on hook resistance, (4) effect of shape of hook on anchorage resistance, semi-circular hooks compared with hooks of changing radius, and (4) to develop a more efficient hook than the Joint Committee 8d. hook. Sixty anchorage specimens 8 by 8 by 16-in. blocks have been tested to date, including load-slip studies on two grades of concrete for both plain and deformed straight bars, two sizes of semi-circular hooks, and compound-circular hooks. Slip is measured internally, at 2 points on straight rods, and at 2 points of tangency on all hooks, by means of Ames dials attached to indicator rods which are welded to the reinforcing bars. The results obtained thus far indicate that (1) the carrying capacity of hooks tested varies directly with compressive strength of concrete; (2) amount of slip varies inversely with compressive strength of concrete; and (3) compound-circular hooks are more efficient than semi-circular hooks. It is proposed to make an experimental analysis of the load-slip relation, including such factors as stiffness of hooks, sliding friction, and distribution of steel stress. Tests are also planned in which the above forms of anchorage will be used in beams.

5—SUGGESTED RESEARCHES

CEMENT

Development of satisfactory method for making accurate particle size separations of relatively large quantities of cement.

A satisfactory method for determining the actual diameter of finely divided particles.

Determination of particle size and size distribution of finely divided materials including cement. Should include study of air analyzer, Noebel elutriator, microscopic projection, or other methods.

Effect of hot cement on temperature and behavior of fresh cement on properties of hardened concrete.

Use of high strength cement in tests of sand.

Review and comparison of foreign and American methods of testing cement.

AGGREGATES

Development of a reliable test for absorption of fine aggregates.

Effect of organic material in fine aggregates on resistance of mortar and concrete to freezing and thawing.

Effect of surface texture or mineral composition of aggregates on transverse, tensile, and compressive strength of concrete.

Effect of heating aggregates and water on strength of concrete.

Volume change of different types of aggregate when subjected to wide ranges in moisture content and in temperature.

Investigation of properties of cinder concrete; influence of cinder aggregate on corrosion of reinforcing.

Methods of determining durability of fine and coarse aggregates.

Methods for testing the quality of sands for use in concrete (shale content, strength, and resistance to weathering and abrasion, etc.)

Effect of coatings on coarse aggregate on strength of concrete.

Effect of grading of fine aggregate on properties other than strength of mortar and concrete.

Effect of aggregate size upon strength, Young's modulus, Poisson's ratio and other properties, with special reference to the larger gradings such as those which result when cobbles are used in mass concrete.

PLAIN CONCRETE

Effect of vibration during setting on strength of concrete.

Effect of temperature at time of test on strength of concrete.

Effect of calcium chloride and sodium chloride on hardened concrete and methods of protection against their action.

Effect of long-continued mixing on strength of concrete.

Influence of high pressure steam curing on strength, volume change, and resistance of concrete to corrosion in sulfate solutions.

Study of fatigue of concrete under (a) slowly applied loads, (b) impact, (c) high frequency vibration.

Effect of high and low temperatures on strength of concrete at different ages.

Investigation of the uniformity of concrete in deep sections when wet mixtures are used.

Study of the influence of moisture content at time of test on flexural, compressive, and tensile strength of specimens of different size.

Strength relations for different types, shapes, and sizes of mortar and concrete specimens at different ages.

Development of methods of making absorption tests of concrete products.

Study of the durability of finish coat stucco mixtures.

Effect of admixtures such as inorganic silt or clays, diatomaceous earth, etc., on strength, bond, permeability, workability, yield and resistance to weathering of concrete.

Temperature in concrete slabs as influenced by curing methods.

Effect of various factors on volume changes due to (a) temperature, and (b) moisture changes.

Study of deformation or flow of concrete under sustained load.

Tests to determine workability of concrete.

Study of effect of admixtures on workability of concrete.

Simple method of designing concrete mixtures which will recognize that the "character" as well as the grading of the aggregate affect the strength of the concrete.

Laboratory tests for determining the resistance of concrete to weathering.

Effect of percolating water on hydraulic structures.

Study of maximum temperatures attained near centers of concrete masses.

Extent and effect of shrinkage cracks in arch dams.

Air deterioration of concrete in different humidities.

Moisture gradient through masses of concrete.

Magnitude of segregation encountered when concrete is discharged from paving mixer.

Study of internal friction in concrete.

Development of a practical consistency regulator.

Study of causes of scaling of concrete.

Field study of joints in road slabs to determine types least susceptible to spalling due to curing. Particular attention to be given to behavior of dummy joints.

Study of methods of controlling movement of slabs—similar to rail creep.

Determination of that grading of sand and gravel which when used as concrete aggregates and designed by the water-cement ratio method will give the least cost per cu. yd. of concrete.

REINFORCED CONCRETE

Study of the effect of coatings and admixtures on the bond resistance of concrete.

Investigation of the initial stresses developed in steel and concrete due to shrinkage, and their effect on the stresses in flexure.

Study of shearing stresses in webs of shallow beams and methods of web reinforcement.

Effect of mortar coating on reinforcing bars on bond resistance.

Effect of repeated and continued loads at high stresses (1) on bond, and (2) on direct compression.

Bond tests of high early strength concrete.

Initial stress developed in steel and in concrete due to setting.

Effect of curing water on reinforcing steel and depth to which such water permeates reinforced concrete members.

Study of column splices, tied, spiralled, combination, and composite.

Bond—hooks vs. straight bars, radius of hook, etc.

Study of behavior of reinforced concrete arches during and immediately following construction.

Shearing stresses due to concentrated loads on floor slabs.

Effect of concentrated load on a cantilever slab with particular application to most economical design of bridge of width just sufficient to permit passing of two vehicles.

Readers are referred to the JOURNAL *for May, 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931.*

Discussion of Paper by Lars Jorgensen

Memorandum on Arch Dam Developments*

BY FREDRIK VOGT†

The author is to be congratulated for his pioneer work on the development of arch dam construction.

As to the safety of single span arch dams, the records are rather convincing: No one has failed, while a great number of dams of other types have failed for different reasons. It may be objected that the Moyie River and the Lanier Lake arch dams failed,[1] but what the records really tell, is, that the proper arch dams did not fail in spite of the failure of a gravity abutment wing in one case and of a faulty rock abutment in the other case. The lesson is therefore not that the arched structure was weak, but, on the contrary, that in this case it was strong enough to carry the water-load even though the regular support suddenly broke away for half the height at one side.[2] In addition, the failure repeats the old lesson, that a gravity dam on a faulty foundation is a dangerous structure.

Some problems in dam construction are common for all types of dams, which is especially true for the problem of how to make permanent and impermeable concrete. It may in this connection be worth while to refer to the comprehensive investigations carried out by the Norwegian Engineering Society on disintegration of concrete in dams, particularly when exposed to water with small silt content, but high acidity, as very often is found in high mountain and sub-arctic climates.[3] The self-tightening of the concrete referred to by the author, and well known for dams

A. C. I. Journal, Sept., 1930; *Proceedings*, V. 27, p. 1.
†Docent at the Norwegian University of Technology, Nidaros, Norway.
[1]*Eng. News Rec.* 1926, II., p. 616.
[2]*Transact.* A. S. C. E., 1929. p. 1273.
[3]Den Norske Ingeniörforenings Betongkomite *Meddelelse nr. 1*, Oslo, 1930.

in southern climates, can not be counted upon when it is exposed to this type of water. On the contrary, all seepages are then slowly increasing and the cement correspondingly dissolved, if seepages exist.

In addition to this problem, the arch dams have their own problems, namely the analysis of stresses and the design of the dam according to this analysis. Not many special problems in civil engineering have been so much discussed during the later years, as the analysis of stresses in single span arch dams. Most of the investigations, however, have been merely mathematical analyses, and the problem goes deeper than to a clever application of mathematics. A mathematical analysis is indispensable, but it can only give the consequences of certain assumptions, and positive information on what assumptions have to be made is still more indispensable. It is a well known fact that tensile cracks can be observed in structures where the usual theory calls for compressive stresses, and also that some of the earlier mathematical studies on arch dams indicated unavoidable horizontal cracking due to vertical tension at places where no cracks or indication of cracks are observed. If something is wrong, it is hardly the mathematics in itself, but the assumptions on which it is based.

Two main problems, which can not be said to be conclusively solved at present, are: (1) What is the time effect in continuously stressed concrete, and, (2) What are the shrinkage and swelling (volumetric changes) in concrete of dams, including volumetric changes due to chemical changes, temperature changes, and moisture changes, and how are these volumetric changes distributed over the section.

As to the time effect, or flow, much basic information is obtained particularly by Professor R. E. Davis[4], and his results are confirmed by tests at the University of Colorado.[5] The main result seems to be that the flow of concrete under continuous load is about proportional to the stresses, at least up to twice the working stresses. Hence the flow does not change materially the distribution or magnitude of the stresses due to load, but it

[4]"Modulus of Elasticity and Poisson's Ratio for Concrete," *Proc.* Am. Soc. Test. Mat., 1929, Part II.

[5]Gilkey and Vogt, discussion on this paper, loc. cit.

reduces the magnitude of the stresses due to volumetric changes.[6] For wet concrete this reduction seems to be about 40 per cent for slow changes, for dry concrete the reduction is much greater. These results, however, need to be confirmed, and the tests need to be extended to different kinds of concrete and stone under different conditions. Also the effect of variable loads should be investigated, before the problem can be characterized as solved.

Laboratory tests have given information on the volumetric changes which can be expected under certain conditions, but before these results can be used in the analysis of large dams, it is necessary to know the conditions in the interior of the mass. The temperature changes in the interior can be estimated with fair accuracy, but the moisture changes differ considerably from one case to another, hence also the actual magnitude of the shrinkage is variable. In a great number of dams the writer has found variations in shrinkage from 0 to 0.02 per cent according to type of concrete, method in concreting, and climate. Before a stress-analysis can be claimed to be fairly accurate, this matter has to be investigated very carefully. The writer has tried to compile information on this subject,[7] but a cooperative work by all interested parties is needed before the matter can be solved.

The author has introduced grouting of the joints in order to eliminate at least partly the effect of the volumetric changes. If the reservoir is filled before grouting, the dam is exposed to shrinkage stresses, and if the joints are grouted too early, the subsequent shrinkage will produce shrinkage stresses in the final state of the dam. For large dams the radiation of the initial heat demands several years, and if the dam must be put in use before the radiation is completed, the choice of a proper time for grouting is a problem which only can be solved by means of intimate knowledge of the volumetric changes. Furthermore, changes in temperature and moisture content, and in the distribution of these, go on with the times, and cause volumetric changes, which can not be counteracted by grouting. The problem of volumetric changes should therefore not be neglected because of the grouting.

[6]Gilkey and Vogt, discussion "Plastic Flow in Concrete Arches," *Proc.* A. S. C. E., 1930, p. 1440.

[7]F. Vogt "Shrinkage and Cracks in Concrete of Dams" to be printed in English this winter in *Det Kgl. Norske Videnskabers Selskab*, Skrifter, Nidaros, Norway. See also F. Vogt "Fundamental Views in the Analysis of Arch Dams," World Power Conference, Berlin, 1930, paper No. 159.

In the later years the Bureau of Reclamation has provided for grouting of the joints in their arch dams. Also the writer has tried to develop the grouting one step farther by introduction of a "multiple pressure grouting system," by which initial stresses in the structure can be introduced in such a way that they partly counteract the stresses from dead load, water load and shrinkage, thus reducing the final resulting stresses[8].

If the arch elements can carry the total water load without being overstressed, an arch dam must be considered as very safe against failure due to loading. However, in many cases the vertical stresses are higher than the horizontal arch stresses, and vertical tension may develop horizontal cracks, which invite the water to attack the concrete chemically and mechanically. Also the investigation of these vertical stresses is therefore of importance.

For dams in narrow canyons the vertical stresses are usually not the limiting factor. For instance, for a preliminary design for Diablo Canyon dam, the writer in 1927, then employed by the author, computed the vertical stresses at the base of the maximum section as 201 — 135 = 66 lb. per sq. in. upstream, and 247 + 291 = 538 downstream compression due to dead load and water load. The final design will show slightly smaller stresses. Assumed that the joints are grouted at low temperature after the major part of the shrinkage has taken place, the volumetric changes will not aggravate the result enough to make these stresses dangerous.

For dams in wide canyons the ratio of radius to height will be larger, and then a vertical tension at the upstream face is harder to avoid. The Stevenson Creek Test Dam is one example of this. The dam cracked horizontally at the base with water level between half and two thirds of the height. Later on it cracked vertically and horizontally at other places. This test dam has exceptionally daring dimensions in order to obtain large deflections and eventually get a test to failure, and a direct comparison with commercial dams is therefore difficult. Some of the most daring dams may be assumed cracked in a similar manner at the base, but for a number of arch dams, conclusions about

[8]See "Hydraulic Engineering," Los Angeles, Dec., 1928.

such cracks made from mathematical analysis are not checked by any positive indications of cracks in the actual structure. This discrepancy between observation and analysis may be due to wrong assumptions with respect to shrinkage and swelling, or due to incomplete analysis, or both. A number of factors, which formerly were neglected in the analysis, have proved to reduce the vertical tension which otherwise could have been expected, and a complete analysis must take into account not only the deformation of arch and cantilever elements due to thrust and bending, but also shear deformation, elastic yielding of the foundation, lateral deformation, and stiffness against tangential displacements.[9] In some cases perhaps also the stiffness against twisting of the elements may be of importance.

The author has several times also pointed at the release in the vertical bending stresses due to swelling at the upstream wet face and the following warping of the dam. This factor will undoubtedly sometimes be of importance, but as a result of observations on dams, the writer has received the impression that this factor has sometimes been overemphasized, and that no release worth mentioning can be counted upon except for the very skin.

The maximum distance, which can be used between the joints if interjacent cracks should be avoided, must necessarily be dependent on the type of the dam, and the author can hardly expect to find "any spacing which he thinks is the best in general" given by a certain number of feet. If the shrinkage were the same and distributed in the same way for a large dam, and for a model of this in scale $1:n$, the stresses would also be the same in dam and model. As an example, may be mentioned that the shrinkage cracks at both abutment joints in Stevenson Creek dam were truly repeated in a mortar model in scale 1:12. If cracks can be avoided in the dam by a maximum spacing a of the joints, then the spacing in the model should be maximum $a:n$. If dams with similar shaped cross-sections were divided in sections by joints with spacing given as a certain fraction of the height for each block, each block would be a model of all the other blocks, and the same shrinkage in all dams would give the same stresses,

[9] "Fundamental Views, Etc.," loc. cit.

i. e., the proper spacing should be a certain fraction of the height. To some extent this rule may be correct, the writer has, for instance, noticed cracks in all sections midway between joints with spacing 30 to 40 ft. on a small arch dam at Twin Lake, Cal., while large dams usually do not crack if the spacing is 50 ft.

The volumetric changes are, in general, greater for small thin-sectioned structures than for thick-sectioned due to more complete drying, and the spacing of the joints should then possibly be increased for large dams more than in proportion to the height. This, however, is in disagreement with the experience. The reason may be that for large thick dams the volumetric changes in the interior mass is much smaller than for concrete near the face, and that cracks are started at the face due to this difference more than due to the restraining from the foundation. Regardless of the size of the dam, a certain maximum spacing should therefore not be exceeded, but this limit will necessarily be dependent on type of concrete and on climatic conditions at the dam-site, and no simple rule will hold for all locations.

One experience is very frequently repeated, namely that contraction joints are continued down to the very foundation by irregular cracks, if the joints initially stopped at a higher elevation. As a rule all joints should therefore be continued directly down to the very base at once, in order to avoid this cracking.

BY W. NIMMO*

Since there is much truth in the old adage that "an ounce of practice is worth a pound of theory," this paper, in which the author sets forth the results of his long experience in the design and construction of arch dams, is of particular value. Little real progress in the determination of stresses in an arched dam was made until Professor Cain published well known equations which have become the basis for much subsequent work. Numerous attempts have been made to analyze the stresses in an arch dam by equating the deflections of arch slices and cantilevers at one or more points, and, while many elegant mathematical treatments have been evolved, few of them are of real practical use.

The trial load method,[1] developed by practical designers, has

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[1] "Analysis of Arch Dams by the Trial Load Method" by C. H. Howell and A. C. Jaquith, *Trans.* Am. Soc. C. E., V. 93, 1929.

been used to a considerable extent for the design of large dams and the method appears to the writer to be the most suitable of the theoretical treatments, and some arch dams in Europe have been successfully designed by somewhat similar methods. The experimental results obtained from models and from the Stevenson Creek dam show that, at least for the conditions obtained during such experiments, the trial load method yields results which are in close agreement with observation. Most of the published deflection curves for actual dams, including those recently obtained from a number of Swiss dams,[2] exhibit the same general characteristics as those derived from the experimental work.

Notwithstanding these results such an experienced designer as the author, while admitting the existence of cantilever action, considers that its effect is so uncertain that he prefers to neglect it in his analysis of stresses and assumes that all the water load is carried by arch action or in the lower portion of the dam by shear. In the Stevenson Creek and Swiss experiments the variation of water level was so rapid that the effects of water soaking and plastic flow are practically eliminated, but this is not so for a dam in ordinary use. Expansion of the upstream face due to water soaking will appreciably lessen the load supported by the cantilevers, but the effect of plastic flow is less certain. Deformation of the foundation, including movement downstream and tilting at the base and spreading of the abutments at the higher levels and also cracking due to shrinkage was clearly in evidence at Stevenson Creek but their extent in a large dam is still a matter upon which reliable data is lacking. The effect of deformation of the foundation will be to diminish the load carried by the cantilever while shrinkage, if it be excessive, may crack the cantilevers and almost destroy their capacity to carry load. Moreover the dams dealt with by the author have been pressure grouted after shrinkage has taken place and, since the full water load on them usually occurs during the summer, the deflections under load will be less than in the case of those dams which have not been pressure grouted, and this will further decrease the proportion of load that can be carried by the cantilevers.

[2] "Deformationsmessungen an Staumauern nach den Methoden der Geodäsie" by W. Lang *Service Topographique*, Berne, 1929.

The dams that have been designed by the trial load method have not, so far as is known to the writer, been pressure grouted and therefore cantilever action may be expected to play a large part necessitating its inclusion in the analysis. The fact that grouting throws more of the work upon the arch is, in the opinion of the writer, a great advantage. In the southern portion of Australia dams are usually filled during the colder portion of the year so that the maximum downstream deflection due to temperature occurs at the same time as that due to load and under these conditions to minimize cantilever action it would be necessary to employ a sufficient pressure in grouting to force the empty dam somewhat upstream.

It would be of great value if deflection measurements extending over several seasons were available for a large arch dam that has been pressure grouted and one of similar type that has not been so treated. It is remarkable that although computation indicates the existence of tension amounting to 289 lb. per sq. in. in the Lake Spaulding dam, no harmful result has been observed.

The author's satisfactory experience with a mat of rocks as an energy dissipator under high overfalls is of interest, but in this country where masonry dams are usually built in sandstone formations, the method would probably not give sufficient protection against wear.

AUTHOR'S CLOSURE

It has been well stated by both W. Nimmo and Fredrik Vogt, that certain difficulties exist, when it comes to deciding upon proper values to use for modulus of elasticity, shrinkage and water soaking effect when designing arch dams.

More tests have been made to find values for the modulus of elasticity and shrinkage than for the water soaking effect. The main reason may be that the distribution of this effect is much more difficult to find than the other properties of the concrete, but it is at least as important, if one desires to attempt to distribute the waterload between arch and cantilever.

If the design has been based upon a certain calculated distribution of the load between arch and cantilever and this distribution is wrong, "It is hardly the mathematics itself, but the assumptions on which it is based" that is wrong, to quote from Dr. Vogt.

The writer prefers to use higher arch stresses and assume that no load goes on the cantilever, except near the bottom. There have been no tests made on commercial arch dams, as far as the writer knows, where it could be shown, that the cantilever took a certain definite part of the load relieving the arch of just that much. The Stevenson Creek tests may show a certain division of load, but the load in this case came on almost instantaneously and besides the proportions of this dam are not comparable with those of a good commercial arch dam.

Not alone is the sustained modulus of elasticity only about half of the instantaneous (at about 600-lb. compression per sq. inch in air), but the E for sustained load is from 60 per cent to 63 per cent higher for wet than for dry concrete according to tests made by Professor R. E. Davis in connection with the concrete in the Stevenson Creek dam (page 213 *Proc.* Am. Soc. of Civil Engineers, May, 1928, Part 3).

An important item for an arch dam is the grouting of the vertical contraction joints at the right time. The writer prefers to wait until the dam has taken load at least once and has cooled off. Unfortunately the owner will often not wait that long, and on a diversion dam very often grouting can not be put off that long. In such cases grouting will only be say 50 per cent effective and some cantilever action undoubtedly will develop.

It is not intended to grout the Diablo Dam until the Spring of 1932. After that time a power plant will utilize the head behind the dam and the water can not be drawn low enough to relieve the load on the structure sufficient for grouting purposes. By that time the heat should be out of the dam body and the structure should have taken its permanent set. Before grouting has taken place, there will undoubtedly be some cantilever action during the winter with reservoir full, this however, the structure will easily take care of.

The spacing of the contraction joints on the Diablo Dam was approximately 75 ft. This has been shown to be too large, since shrinkage cracks have appeared in the four middle blocks, between the artificial joints, extending from near the bottom to within 80 ft. of the crest. One shrinkage crack goes through the

dam body, the others appear, so far, on the down stream face only. Grouting the artificial joints later is expected to practically close these cracks and the load may do the rest.

For overfall dams on somewhat soft rock as mentioned by Mr. Nimmo it will be the best practice for arch dams to create a lake below the main dam and concentrate the overfall as much as possible into this lake.

In conclusion the writer desires to correct a misprint on page 62 line 17 from below. It should say: T. H. Carver was the principal assistant engineer in charge, etc.

The following is a list of Constant Angle Arch Dams built so far with their principal dimensions given. It can be seen, that there is nothing new and untried about arch dams any more, since some of the highest dams in the world are in this list and have so far given good satisfaction.

CONSTANT ANGLE ARCH DAMS BUILT IN THE PAST

1. Lake Spaulding Dam, South Yuba River, built 1912-13; 295 ft. high above foundation; 275 ft. high above river level; 770 ft. span; 192,000 cu. yd. of concrete; upstream radius at crest 440 ft., at bottom, 250 ft.

2. Salmon Creek Dam, Alaska near Juneau, built 1912-13; 168 ft. above river level; 545 ft. span; 52,000 cu. yd. of concrete; upstream radius at crest 331 ft., at bottom 147.5 ft.; thickness at crest, 6 ft., at bottom 47.5 ft.

3. Kerckhoff Dam, San Joaquin River, California, 1919; 108 ft. high; 450 ft. span; 23,000 cu. yd. of concrete; dam designed for 100,000 sec. ft. free overfall.

4. Carmel River Dam near Monterey, California, 1920; 90 ft. high; 270 ft. span; radius at crest 135 ft., at bottom 80 ft. long; thickness of dam 5 ft. at crest, 16 ft. at bottom; 10,000 sec. ft. free overfall.

5. Bullards Bar Dam, Yuba River, California, 1923; 183 ft. high; 440 ft. span; in connection with power plant. Upstream radius at crest 240 ft. long, at bottom 94 ft.; thickness at crest 6 ft., at bottom 43 ft.; 47,000 cu. yd.; 65,000 sec. ft. free overfall.

6. Lost Creek Dam, Lost Creek, California, 1923; 112 ft. high; 380 ft. span; radius upstream crest 200 ft., at bottom 90 ft. long; thickness at crest 4 ft., at bottom 23.5 ft.; 11,000 cu. yd. Spillway free overfall 15,000 sec. ft.

7. Concow Dam, Concow Creek, Calif., Thermolite Irrigation District, 1924; height 90 ft. above stream bed; span 275 ft.; upstream radius at crest 180.6 ft., at bottom 78 ft.; thickness at top 3.75 ft., at bottom 18.9 ft; 1:3:6 concrete; 63,000 cu. yd. of concrete. Spillway 10,000 sec. ft. free overfall.

8. East Canyon Creek Dam, Utah, 1917; 140 ft. high above river level; 190 ft. high above foundation; about 400 ft. span including tangents; upstream radius at crest 98.8 ft. long, at bottom 70 ft. long; crest thickness 5 ft.; bottom thickness 26.25 ft.; 15,888 cu. yd. of concrete. Total overall cost $8.62 per cu. yd.

9. Sun River Dam, Idaho, 1917; 100 ft. high; 250 ft. span.

10. Clear Creek Dam, Washington, 1914-18; 90 ft. high; 180 ft. span; approx. 5,000 sec. ft. free overfall.

11. Manila Dam, Philippine Islands, 1913; 98.4 ft. high; 100 ft. span.

12. La Jogne Dam, Switzerland, 1920; 186 ft. high; upstream crest radius 126 ft.; 250 ft. span.

13. Crocodile River Dam, South Africa Union Irrigation District, 1922; 198 ft. high; 450 ft. crest length; upstream radius at crest, 225 ft.

14. Amsteg Dam, Switzerland, 1921; 105 ft. high; upstream crest radius 62 ft.; 102 ft. span.

15. Gide River Dam, Sweden, 1919; 85 ft. high; 131 ft. span; upstream radius at crest 98 ft; and 72 ft. at an elevation 43 ft. lower down; spillway free overfall.

16. Butte Creek Dam, Calif., 1915; 45 ft. high; 85 ft. span; spillway free overfall.

17. Mormon Flat Dam Salt River, Arizona, 1923-24; height 229 ft. above bedrock; span 326 plus 90 ft. gravity tangent; upstream radius at crest 193 ft. at river level 100 ft.; thickness 8 ft. at top and varies from 20 to 29 ft. at bottom; 43,000 cu. yd. of concrete; 150,000 sec. ft. spillway on gravity section. Total cost of concrete $473,000; 63,000 acre ft. of storage.

18. Gerber Dam, Klamath Project, Oregon; U. S. Reclamation Service, 1923-24; height 70 ft. above river bed, and 82.5 ft. above lowest foundation; span 375 ft.; length of crest along curve 478 ft.; upstream radius at crest 197 ft.; at bottom 155 ft.; thickness 5 ft. at top, and 15 ft. at streambed; 11,880 cu. yd.; 1:2½:5 concrete; 31½ tons of reinforcement; spillway free overfall.

19. Upper Hubbart Dam, Flathead Project, Montana; U. S. Reclamation Service, 1923; height 92.5 ft., above river bed, 113.5 ft. above foundation; span 400 ft.; length of crest along curve 503 ft.; upstream radius at crest 200 ft., at bottom 194 ft.; thickness 5 ft. at top, 22 ft. at bottom; 1:2½:5 concrete; 17,000 cu. yd. of concrete; spillway free overfall.

20. Montejaque Dam, Caduares River, Spain, 1923-24; height 210 ft. above river level, and 273 ft. above foundations; span 208 ft.; length of center line at crest 256 ft.; upstream radius 123 ft. at crest, 72 ft. at base; thickness at crest 10 ft., at base 55 ft.; 35,000 cu. yd. of concrete.

21. Emigrant Creek Dam, Oregon; Talent Irrigation District, 1924; height 100 ft. above stream level; 115 ft. above foundation; span 360 ft.; upstream

radius at crest 165, at base 132 ft.; thickness 5 ft. at crest, and 19.3 ft. at base; 14,000 cu. yd. concrete; spillway partly free overfall.

22. Balch Diversion Dam, Kings River, California, 1926; 100 ft. high above foundation (initial height), ultimate 150 ft.; 300 ft. span; 9,300 cu. yd. of concrete; upstream radius at crest 184 ft. (initial) at bottom 90 ft.; spillway free overfall 40,000 sec. ft.

23. Glines Canyon Dam, Port Angeles, Washington, 1927; 200 ft. high from bottom of foundation to the crest; radius of upstream face at crest 128 ft., at bottom 62 ft.; dam is 28 ft. thick at the base, and 4 ft. thick at top; arch between two gravity tangents 152 ft. long; 16,630 yards concrete in arch; 20,000 cu. yd. total.

24. Big Santa Anita Dam, near Los Angeles; 225 ft. high; 515.0 ft. span; upstream radius at crest 301.5, and 129 ft. long at bottom; thickness at crest 7 ft., at bottom 48 ft.; 76,000 cu. yd. of concrete. Cost $1,211,000, 1927.

25. Urayama River Dam, location near Tokio, Japan, 1921; height 45 ft.; thickness at crest 4 ft., at bottom 9 ft.; radius at crest 48 ft.

26. Lake Cushman Dam, Washington, City of Tacoma; 280 ft. high; upstream radius at crest 200 ft., and 131 ft.; 150 ft. below crest to bottom; span of arch proper 340 ft.; thickness at crest 8 ft., at bottom 45.5 ft.; 85,000 cu. yd. of concrete; finished 1926.

27. Horse Mesa Dam, Arizona; 305 ft. high from foundation to parapet; top upstream radius at crest 251.4 ft., at bottom 82 ft.; thickness at top 8 ft.; thickness at bottom varies from 43 ft. to 57 ft.; stores 245,000 acre ft.; arch span 450 ft.; total span 784 ft.; contains 150,000 cu. yd., 1927. Cost $1,500,000; spillway capacity 150,000 sec. ft.

28. Bucks Creek Diversion Dam, Plumas County, Calif.; 96 ft. high; 420 ft. span; upstream radius at crest 213 ft., at bottom 156 ft.; thickness of dam at crest 6 ft., at bottom 27 ft.; 19,200 cubic yards of concrete; has two gravity tangents, 1927.

29. Canadea Dam, New York; 130 ft. high above river bottom; one gravity abutment arch span 480 ft.; upstream radius at crest 265 ft.; upstream radius at bottom 200 ft., and thickness at crest 5 ft.; at bottom 44 ft.; 50,000 cu. yd. of concrete, 1928.

30. Santeetlah Dam, Tallasse, North Carolina, Aluminum Co. of America; 202 ft. high above foundation; two overflow gravity tangents; arch span 340 ft.; upstream radius at crest 300 ft., at bottom 207 ft.; thickness 10 ft. at crest, and 48 ft. at bottom; 75,000 cu. yd. of concrete in arch portion; 217,000 yards total; total length of dam 1,120 ft., 1927.

31. Pacoima Dam, for Los Angeles County Flood Control; 365 ft. high above river bed; 375 ft. high above foundation; span 560 ft.; upstream radius at crest 340 ft., at bottom 171.34 ft.; thickness of dam at crest 8 ft., at bottom 96 ft.; 215,000 cu. yd. of concrete.

32. Grizzly Creek Diversion Dam, Plumas County, Calif.; 90 ft. high; 420 ft. span; upstream radius at crest 205 ft., at bottom 140 ft.; thickness of dam at crest 6 ft., at bottom 22 ft.; 18,000 cu. yd. of concrete, has gravity tangent; spillway partly overfall.

33. Von Geisen Diversion Dam on Bear River, California; Nevada Irrigation District; 100 ft. high; 365 ft. arch span between two tangents; upstream radius at crest 205 ft., at bottom 184 ft.; minimum thickness 5 ft.; thickness at bottom 22 ft., in the middle of arch. Contract price $157,000, 1928; spillway free, overfall 10,000 sec. ft.

34. Deer Creek Diversion Dam, Nevada Irrigation District; 97 ft. high; 270 ft. span; upstream radius at crest 136 ft., at bottom 98 ft.; minimum thickness 4 ft.; thickness at bottom 12 ft., in the middle of arch. Contract price $52,000, 1928; spillway free, overfall 7,500 sec. ft.

35. Pigeon River Dam, North Carolina; 190 ft. high, plus 10 ft. overflow; 600 ft. span; upstream radius at crest 320 ft., at bottom 244 ft.; minimum thickness 16 ft., just below overflow crest; thickness at bottom 38.5 ft.; Waterville Hydro-Electric Development of Carolina Power and Light Company; 120,000 cu. yd., 1929.

36. Canyon Diablo Dam on Skagit River, City of Seattle; 372 ft. high; span of arch 540 ft., arch butting against gravity tangents; total span approximately 950 ft.; upstream radius at crest 390 ft., at bottom 163 ft.; thickness at crest 16 ft.; thickness at bottom 98 ft.; 316,000 yards of concrete, 1930; overflow on tangents 100,000 sec. ft.

37. The Reeder Gulch Dam for the City of Ashland; 110 ft. high; overflow 415 ft. span; upstream radius at crest 230 ft.; at bottom 150 ft.; thickness at top 7 ft., at the bottom 19.5 ft.; 15,000 cu. yd. of concrete, 1928.

38. Yellow Creek Dam No. 1, Tuscaloosa, Alabama, 1929; 65 ft. high. This dam has horizontal sections of elliptical shape and is symmetrical about the major axis. Semi-major axis 179.5 ft. at crest; 166.5 ft. at base; semi-minor axis 149.26 ft. at crest; 130.62 ft. at base; thickness at crest 5 ft., at base 20 ft.; approximately 7,000 cu. yd. of concrete.

39. The Calles Dam, Mexico, River Santiago, 1929; height of dam 217 ft.; arch with gravity tangents; length of crest 985 ft.; overhang on downstream side at crown 6.66 ft.; radius variable in the same horizontal plane.

40. The Calderwood Dam, Tennessee, 1930; height of dam 230 ft. above foundation; length of upstream radius at spillway level 337.5 ft., 281 ft. at bottom; designed for 200,000 sec. ft. overflow; thickness at top 25 ft., at bottom 48 ft.

41. The Calaveras Dam near Stockton, Calif., 1930; height of dam 160 ft.; arch with gravity tangents; length of crest 1400 ft.; dam 50 ft. thick at base, and 7 ft. thick at top; upstream radius at crest 365.2 ft., at the bottom 204.8 ft.; lowest bid $838,715; 105,000 cu. yd. of concrete; dam overhangs in the middle on the downstream side and at the abutments on the upstream side.

42. The Juncal Dam, Montecito Water District, Santa Barbara County; height of dam 160 ft.; length of upstream radius at crest 196 ft., at bottom 132 ft.; thickness at the crest 6 ft., at the bottom 64 ft., 1930; span 350 ft.; crest length 440 ft.; 34,400 cu. yd. of concrete.

43. Upper Hansen Dam, Los Angeles Flood Control; height of dam 245 ft.; length of upstream radius at crest 190 ft., at bottom 132 ft.; thickness at crest 8 ft., at bottom 62 ft.; arch with two gravity tangents; right tangent spillway 150 ft. long; span of arch portion 265 ft.; left tangent 170 ft. long; 90,000 cu. yds. of concrete; 90,000 cu. yd. of excavation.

44. Dam near Safford, Arizona; 110 ft. high; 130 ft. span; central angle held at about 130 degrees; upstream radius at top 80 ft., at the bottom 14 ft.; dam 2 ft. thick at the crest, and 3 ft. thick at the base at the crown, thicker at the abutments; spillway capacity 1000 sec. ft.

45. Stewart Mountain Dam, Salt River, Arizona; height of dam 212 ft. above bedrock; 1200 ft. long; 117,000 cu. yd. of concrete; radius at top 273 ft.; at the bottom, face is irregular; thickness at top 8 ft., at bottom 33 ft. minimum.

46. Dam at Sautet on the Drag River near Grenoble, Switzerland; 443 ft. high, with secondary dam 180 ft. high below. Dam is designed for unbalanced pressure of 430 ft. (130 meters) head. Crest length 246 ft. Subtended angle held substantially at 90 degrees. Upstream radius at bottom 76.6 ft.; thickness at the bottom 54 ft. Dam contains 74,500 cu. yd. of concrete. Under construction.

47. Ariel Dam on the Lewis River, Wash. 300 ft. above lowest bedrock. Upstream radius at crest 400 ft.; at bottom 50 ft. Dam 93.5 ft. thick at base and 18 ft. thick at crest. Spillway on gravity abutment; capacity 120,000 second feet of water. Arch part of dam 760 ft. long at crest. Approx. 300,000 cu. yd. of concrete in complete dam. Under construction at present.

Discussion of Report of Committee 506

Proposed "Construction Specifications for Concrete Work on the Small Job"

BY H. N. WALSH*

I suggest the insertion of a paragraph to the following effect immediately after the descriptions of the three classes of concrete on page 81:

Where rapid hardening portland cement is to be used, compression tests shall be made in an approved laboratory, to determine the water-cement ratio that will give concrete of the required strength at the age at which the concrete is to be put into use.

My experience has been that the relation strength to water-cement ratio may vary considerably for rapid hardening portlands, even for different shipments of the same brand. Further, as concrete made with these cements is generally put into service after 7 to 10 days curing, and as the strengths at the early ages are most affected by excess water, there is more necessity for careful determination of the proper water-cement ratio when rapid hardening cements are being used.

In the paragraph on page 81 dealing with water carried by sand, a warning might be added that if fine sand is used (being possibly the only fine aggregate economically available), then the job test for water content should be made obligatory. A fine sand may carry as much a 0.5 gallons of water per cu. ft., when appearing to be only slightly damp.

Many architects and engineers might like more detailed description of the consistency than that given on page 85; they would probably welcome reference to the slump test to indicate the approximate consistencies suitable for different classes of work. I, therefore, suggest consideration of the inclusion of a table on the following lines:

If the slump test is used, the concrete for different classes of work shall have the following slumps:

*Professor of Civil Engineering, University College, Cork, Ireland.

Mass concrete	2 in.-4 in.
Decking with little reinforcement	
Heavy sections of reinforced concrete	
Thin vertical members	5 in.-7 in.
Thin horizontal members	7 in.-9 in.
Roads	1 in.-3 in.

or as otherwise specified by the Architect or Engineer.

I am aware of the dangers attendant on the inclusion of such a list of slumps. Very probably it has been considered and decided against by the Author. Nevertheless I would like to have the matter discussed.

The amounts of the slumps stated above are merely tentative.

BY A. R. ELLIS*

The author of the specifications, so far as I can see, does not define the "small job." Some small jobs are quite important, and in some of them the quality of the materials involved are consequently of greater importance than the quality of materials on some large jobs.

On Page 71, under Article 4, the author provides that the cement shall be "certified by the dealer to comply with all requirements of the standard specifications, etc." Dealer certification in some instances is apt to be influenced by the desire to make a sale.

Frequently, it is the small job contractor who is imposed upon by dealers with cement that has been in storage sufficiently long to cause its deterioration. Some dealers do not know when deterioration, due to age, begins, and again the desire to make a sale is likely to influence covering up this fact, even if it is known.

Considerable stress is laid on fine and coarse aggregate and even that water from the usual drinking supply shall be used and then the door left pretty wide open when it comes to cement.

I would like to see this reference to dealer's certification of the cement omitted from this specification.

BY J. D. STODDARD†

Article 4 provides that cement shall be "certified by the dealer to comply with all requirements of the Standard Specifications of the A. S. T. M." We consider this to be a dangerous clause and

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one which should not be permitted to appear in any specification or building code. While it is intended to apply to "small" jobs only, the word "small" is a very flexible one and depends largely upon the opinion of the individual or possibly the viewpoint of the individual as to whether it should be considered small or large. Our experience with "dealers" has been such that we do not believe any great amount of value should be attached to any certification of the quality of portland cement purchased from them. Very few dealers have more than a smattering of knowledge regarding the nature of portland cement or the conditions under which its integrity is preserved or under which it deteriorates, and certainly the certification of quality should not be placed in the hands of such individuals.

Outside of the commercial phases of our activities which we are of necessity compelled constantly to recognize we are interested in the development of the construction industry along wise and safe lines. We trust you will appreciate our interest in the clause referred to, and hope the language will be changed and no such specification be adopted or recommended for use.

Readers are referred to the JOURNAL *for May, 1931, for discussion which may develop. Such discussion should reach the Secretary by April 1, 1931. The proposed specification will be open for discussion at the 27th Annual Convention when the committee will move its tentative adoption.*

ABSTRACTS

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of the current literature of cement and concrete

PREPARED UNDER THE DIRECTION OF THE
AMERICAN CONCRETE INSTITUTE
COMMITTEE ON ABSTRACTS

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F. E. RICHART (*Engineering Design*)
REXFORD NEWCOMB (*Architectural Design*)
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MATERIALS

ADMIXTURES

Densifying of mortar and concrete against dampness and water under hydrostatic pressure. HUGO LEUBE. *Betonwerk* (Germany), May 18, 1930, V. 18, No. 28.—The use of densifying admixtures discussed. Admixtures for this purpose are divided into three classes, namely: liquid, paste and powder. Each is analyzed as to proper mixing technique.—H. FRAUENFELDER

AGGREGATES

An accelerated soundness test. DR. HERBERT F. KRIEGE. *Civil Eng.*, Nov., 1930, V. 1, No. 2, p. 120.—Detailed consideration is given to sodium sulfate soundness test as applied to concrete aggregate (fine and coarse), filter bed sands, and coarser mineral aggregates commonly employed as sewage disposal media. Although this test dates back to 1828, it has not been fully or uniformly controlled. The essential elements in the test are outlined and explained and detailed procedure is suggested. Of thirteen cooperating laboratories, the eight laboratories that followed suggested procedure closely, correctly classified ten kinds of slag and stone into sound, unsound, and questionable groups. Much of suggested procedure represents a consensus of opinion from the answers to 99 questionnaires sent to prominent engineers in the fields of highway, sanitary and testing engineering.—H. J. GILKEY

Gasoline tractors and trailers in limestone mine. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 64.—Centropolis Crusher Co., 8 miles from Kansas City, Mo., mines limestone by room and pillar method. Rooms are 30 ft. wide with 25-ft. pillars. The level is such there is no upgrade to the crusher. Air hammer drills put in 16 ft. holes which are loaded with 40 per cent powder. Top cut is taken and remaining bench is lifted. The working height is 16 ft. Loading is by 1-yd. shovel with 12-ft. boom and 7-ft., 8½-in. dipper stick. It is equipped with an electric light outfit including a 250-watt floodlight. Haulage is by two tractors and side-dump trailers. Distance from face to crusher is about 500 ft. Two trailers deliver 1,200 tons per 10-hr. day and gasoline consumption is about 80 gal. Between shifts tractors and trailers haul fines to dump. Direct charges for labor, fuel and oil approximate 3¢ per ton. The crushing plant, of conventional design, produces these sizes: 2-in., 1¼-in., ¾-in., ½-in., buckshot and fines. The output is sold in considerable part to make ready mixed concrete.—EDMUND SHAW

Influence of dust content in aggregates on weight per cu. ft. and compressive strength of concrete. G. KATHREIN. *Oesterreichische Bauzeitung* (Austria), Oct. 25, 1930, V. 6, No. 43, p. 705-7.—Additions of limestone powder of different finenesses were made to aggregates, and their influence on mortar and concrete properties examined. High early strength cement was used for preparation of 2¾-in. cubes of 1:5 cement : sand mixture with additions of 4, 10 and 20 per cent limestone powder of 2 different finenesses. Compressive strength and weight per cu. ft. were determined after 7 days air storage. It was found, that additions of stone powder have favorable influence on mortar strength, which increases about 50 to 85 per cent with additions of 4 and 10 per cent. Addition of 20 per cent lowers strength. Fine powder has greater influence than coarse particles. Weight per cu. ft. shows similar behavior, increasing with 4 and 10 per cent and decreasing with 20 per cent additions of stone powder.—A. E. BEITLICH

Standardization vs. discrimination. STANLEY HANDS. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 46-48.—The practice of proportioning concrete

by a definite grouping of a limited range of sizes of aggregates appeals to the engineer because it relieves him of much detail and he is not concerned with losses of production. Fact that other methods of proportioning would produce concrete of same quality is not important to him. If, however, he has to produce his own aggregate he will adopt methods of proportioning not based on accepted standards but on natural laws covering use of aggregates as he may find that his pet proportions create undue waste. Discrimination in this case will be favorable to more use of run-of-the-pit. Standards may change but fundamentals and natural laws do not change. The application of fundamentals may make it undesirable that some things should be standardized. Some concrete workers think that the art is working toward discrimination in use of materials. Most of fundamentals are ready. Only cooperation of producer is needed. Utility of aggregates is not to be considered from standpoint of *what* they will make alone but *how* they will make what is intended. Measure of their utility lies in profitableness of their use. The surface property of an aggregate may be an important factor. It is generally known that some surfaces become covered with a gelatinous film that is often transparent and not easily detected. Material which may qualify under grading and sizing standards may be found undesirable for lack of bonding quality. Research has not determined all conditions and proportions that make differences in materials. Until adequate facts are known it would be undesirable for the industry to set up standards. All aggregate associations have exceptional research organizations and it is very desirable that their work be continued.—EDMUND SHAW

Effect of flat and elongated particles in mineral aggregates. STANTON WALKER. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—The purpose of the report is to cite whatever of definite information is available on the effect of flat and elongated particles and to suggest procedure for investigations. Although specifications have commonly drawn discriminatory lines against such material, there are little data upon which to base definite requirements. Research conducted by the National Crushed Stone Association and the National Sand and Gravel Association indicate that flat and elongated particles in the amounts to be expected in practice (10 to 15 per cent) do not have deleterious effects upon workability, strength, finish and quality of surface of concrete. Some tests reported by Gilkey on specially prepared aggregates where the coarse aggregate consisted entirely of flat particles indicated some weakness. It is concluded that further investigations are required before generally acceptable conclusions can be drawn and suggestions for procedure are made.—HIGHWAY RESEARCH BOARD

Uniform description of graded materials and evaluation of their fineness. M. SPINDEL. *Tonind. Ztg.* (Germany), Nov. 2, 1930, V. 54, No. 88, p. 1385-6; No. 89, p. 1401-3.—Surface areas of aggregates are constant for any given fineness modulus so that any change therein gives a direct measure of work performed during grinding. A metric fineness modulus is proposed with the sieve openings 10^0, 10^1, 10^2 . . . 10^6 microns as basis with each range divided into five fractions with intermediate sieves the log. of whose openings shall be 1.0, 1.2, 1.4, 1.6 . . . 5.0, which then correspond to the fineness moduli of gradings of broken solids whose mean sizes correspond to the several sieve openings. This system has the advantage of being applicable to cement and other fine materials.—F. O. ANDEREGG

Bethlehem Mines' newest crushed stone plant. *Rock Products*, Aug. 2, 1930, V. 33, No. 16, p. 33.—At the Steelton plant of the Bethlehem Mines Corp., which produces concrete aggregate, flux and furnace lining dolomite, the sizes of stone to be sold as aggregate are thoroughly washed. Scrubber is 6 ft. dia. and 20 ft. long and capacity of washing plant exceeds 200 tons per hour. The rotary screen has a 6-in. pipe with 52 $\frac{5}{8}$-in. holes for washing the material

received from the scrubber. Additional sprays play on the outside of the screen. Only fresh water is used in the screen and after use there it is sent to the scrubber. The plant is all steel and concrete construction.—EDMUND SHAW

Crushed stone producer noted for originality builds sand plant. *Rock Products*, Oct. 11, 1930, V. 33, No. 21, p. 41-45.—Sand and gravel plant of Dolomite Products Co., Rochester, N. Y., owned and operated by John Odenbach employs an unusual method of handling with a much simplified washing and screening plant. The material is excavated by a 12-in. centrifugal pump on a steel dredge. It is boosted to help it through a 600-ft. pipe line by a 10-in. pump in a permanent concrete building. The full lift is 138 ft. The screening plant has a double-deck vibrating screen below a steel sluice box for the first separations, 2½-in. square meshes on the upper deck and $\frac{3}{16}$-in. square meshes below. Oversize goes to a No. 5 crusher with no return to the screen. The intermediate product goes to a 3-deck vibrating screen to be finished. It has 2¼-in., 1¼-in. and ½-in. meshes and all products go to pockets. The minus $\frac{3}{16}$ in. from the primary screen goes to automatic sand cones. The pockets for holding material are of concrete set below the ground level. They are discharged by a high speed travelling crane with 2½-yd. clamshell bucket, the runway being 40 ft. above rails. Bucket discharges into ten 65-ton steel bins for truck loading or into 3 compartment, 150-ton batching bins. Only steel and concrete are used in construction.—EDMUND SHAW

Light concrete for construction of 400 dwelling houses in Berlin-Zehlendorf. (See FIELD CONSTRUCTION—BUILDINGS.)

Reinforced concrete construction. Part I. Raw materials and their production. (See FIELD CONSTRUCTION.)

Relation between durability of concrete and durability of aggregates. (See PROPERTIES OF CONCRETE.)

CEMENT

Discussion on "A tentative specification for high alumina cements." *Structural Eng.* (England), Nov., 1930, V. 8, No. 11, p. 380-384.—The tentative specification which appeared in Oct. 1930, issue of Structural Engineer is discussed in considerable detail.—V. P. JENSEN

Portuguese standard specifications for cement. *Tonind. Ztg.* (Germany), Oct. 16, 1930, V. 54, No. 83, p. 1319.—Cements of "lower strength" must have at least 2850 and 243 lb. per sq. in. after 7 days water storage and 4300 and 300, compressive and tensile respectively, after 28 days in water. Cements intended for use in sea water must meet the above requirements when stored in sea water and must have at least 5000 and 330 lb. per sq. in. after 84 days similar storage.—F. O. ANDEREGG

Ashes absorbed by cement clinker. H. MOSCHITZ. *Tonind. Ztg.* (Germany), Nov. 13, 1930, V. 54, No. 91, p. 1430-1.—Dust removed from a cement kiln working with thickened slurry was about 3 per cent, most of which escaped from the stack. The clinker takes up about 40 per cent of ash.—F. O. ANDEREGG

Soluble salts in portland cement. *Betonwerk* (Germany), April 13, 1930, V. 18, No. 22.—The efflorescence sometimes noted on concrete and cast stone is discussed and its formation described. Cause is given as water soluble salts in cement. Method of determining these salts in cement is given and a suggestion for preventing efflorescence by use of barium oxide, which transforms soluble salts into insoluble.—H. FRAUENFELDER

German standard specifications for portland cement, iron portland cement and blast furnace cement. *Tonind. Ztg.* (Germany), Oct. 9,

1930, V. 54, No. 81, p. 1292-6.—These 3 cements are placed on a par, definitions being given for each. The composition of slag from which the two latter are ground with portland cement clinker is defined: $\frac{CaO + MgO + 1/3\ Al_2O_3}{SiO_2 + 2/3\ Al_2O_3} > 1$ Fineness is 2 per cent on No. 30 (75 mesh per in.) and 25 per cent of No. 70 (176 mesh per in.). The initial set shall be not less than one hour. The strengths 1:3 with German normal sand and with 8 per cent water must be at least:

Lb. Per Sq. In.

	Standard Cement		High Early Strength Cement	
Storage	Tensile	Compressive	Tensile	Compressive
3 days in water	...	....	357	3570
7 days in water	257	2570	...	....
28 days in water	...	3930	...	....
28 days combined	430	5000	570	7150

The tensile test is not regarded as of as much value as the other. Full directions are given for methods of testing.—F. O. ANDEREGG

Perchloride acid method of silica analysis—Its advantages over double evaporation. WALLACE K. GIBSON. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 70.—The perchloric acid method takes only 30 to 45 m., according to the water that has been added to the solution, saving much of time that has to be spent on double evaporation of silica and further baking of dry residues. Care must be taken not to boil with ordinary dilutions as an explosion will occur before the boiling temperature is reached. If it is desired to boil, the solution should be about 150 cc. with a 0.5 g. sample. The article gives the perchloric acid method in detail with figures showing the silica contents found by this and other methods.—EDMUND SHAW

Improved grinding methods at the Diamond Portland Cement Co. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 51.—At the Diamond Cement Co.'s plant at Middlebranch, Ohio, three 7 x 24-ft. compeb mills were formerly used in grinding raw material. One of these was put in closed circuit with a 14-ft. Sturtevant air separator and after a year's trial of this combination a second mill was similarly equipped. These two mills were then able to grind all the raw material needed. Without air separators each mill produced from 17 to 18 tons per hour. With air separators each produced 22 to 23 tons per hour. Material going to the separator from the compeb has 52 per cent minus 200 and 73 per cent minus 100. The finished product removed by the separator has 88 per cent to 89 per cent minus 200 and 97 per cent minus 100. Returns to the mill from the separator have 15 per cent to 16 per cent minus 200 and 46 per cent minus 100. Total circulating load is 46 tons per hour. Each separator takes 35 h. p. to drive. They are fed by steel encased elevators and the rejects are returned to the mill mixed with the primary feed.—EDMUND SHAW

Further developments in the application of dry fine grinding with air classification. HOWARD G. WRIGHT. *Rock Products*, July 19, 1930, V. 33, No. 15, p. 57.—It has been ascertained that it is more important in preparing material for cement manufacture to eliminate coarse oversize than to produce much "flour." Air classification with grinding is shown to give excellent results in holding the particle sizes within narrow limits. Separation made by reversed current system of air classification used with Hardinge mills is in three stages. Air blown into the mill through a trunnion is reversed and drops the coarsest particles in the mill. The same air issuing from the mill passes through a rotary separator where the medium-coarse particles are removed. Then it passes to a "superfine" air classifier which takes out all but the product desired, this being caught in a cyclone. Oversize particles from rotary and from "superfine" classifiers falls into air tube through which air is blown into mill

and are thus returned to the mill. Records show that power consumption is not much if any greater with air separation than without, added output of the mill making up for the power used by the classifier. A lower ball consumption has been noted. A feed of uniform quality is necessary for best results and installation of a constant weight feeder in one case gave 2.5 tons per hour added mill product.—EDMUND SHAW

Tentative specification for high alumina cements. Prepared by the Institution of Structural Engineers. *Structural Eng.* (England), Oct., 1930, V. 8, No. 10, p. 369-373.—Specification includes method of manufacture and composition of the cement, samples for testing, tests of fineness, chemical composition, compressive strength, setting time, and soundness. Sections deal with non-compliance of tests, copies of vendor's tests and analyses, and the Appendix gives a tensile test for acceptance purposes only.—V. P. JENSEN

Weight of slurry. E. TAYLOR. *Rock Products*, Nov. 8, 1930, V. 33, No. 23, p. 64.—The author shows how he derives the following formula for the weight of one cubic foot of wet slurry: $W = \frac{6243\ s}{100 - a + sa}$ in which W is the weight in lb., s the specific gravity of the dry slurry and a the percentage of water in the slurry.—EDMUND SHAW

Free lime, constancy of volume and strength. Report of research laboratory of the Association of German Eisenportland Cement Companies, Duesseldorf (Germany). A. GUTTMANN. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1078.—Based on own experiments, author agrees in general with findings of G. Haegermann (c. f. *Zement*, Oct. 16, 1930, V. 19, No. 42, p. 982-4) concerning relation between volume constancy and free lime content of portland cement clinker. Presence of free lime has tendency to lower strength of clinker which contains much tricalcium silicate, but is able to improve strength of clinker, which is rich in dicalcium silicate, in later ages.—A. E. BEITLICH

Overcoming heat losses in cement manufacture. GEOFFREY MARTIN. *Rock Products*, July 5, 1930, V. 33, No. 14, p. 42.—The author leads up to a discussion of his own design of a flotation kiln by describing the heat losses in a rotary kiln and comparing with a shaft kiln which is more economical of fuel. He emphasizes that as clinker cannot be made at a lower temperature than 1370°C. (2498°F.), the kiln gases must have a higher temperature to impart this heat. But too high temperature cause high radiation losses. With coal firing a gas temperature of 2600°F. may be obtained. Higher temperatures if they were possible would be more efficient. Costly failure of British experiments at Swanscombe was due to ignorance of these facts. Hot exit gases were introduced into the kiln in an effort to save heat but gas temperature was cooled below 2500°F. and no clinker was formed. The great loss of high grade heat is internal radiation. Various methods to prevent this loss have been suggested and most promising of these is a cloud of spray or dust formed of raw materials in either a wet or a dry condition. These experiments are still going on. Author's flotation kiln, an improvement on design of Sir Percy Girouard, employs dust cloud principle mentioned. The kiln is a cavity worked out of a limestone cliff and it is of such a shape that the gases will circulate vertically. Raw material dust is fed into downwardly flowing side of this circulation and carried to bottom where there is a zone of intense heat. Dust falls through this and becomes clinker. It is discharged by conveyor in tunnel below. Gases after dust has fallen from them rise on other side of cavity and flow across to pick up more dust and descend.—EDMUND SHAW

Contribution to the question of fine cement grinding. ERNST RISSEL. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1079-80.—Two brands of cement were ground to different finenesses and effect of grinding was studied by measuring water permeability of 1:6 mortar test specimens and com-

pressive strength. One fact developed is that fine grinding affects water permeability to a far greater extent than it does compressive strength. Based on this evidence author points out that examination of water permeability must be considered of greater value than compressive strength tests for valuation of cement obtained by fine grinding. Further tests dealing with very fine cement fractions are under way.—A. E. BEITLICH

Influence of mixing water on mortar strength. E. PROBST AND K. E. DORSCH. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1009-11.—Standard compression and tensile strength tests were made with one high early strength cement, two ordinary portland cements, one blast furnace slag cement and one high alumina cement. Mixing water was in one case ordinary service water and in another case distilled water. Samples mixed with distilled water showed higher compressive strength and lower tensile strength than samples mixed with service water. Variations are different for different kinds of cement. Standardization of mixing water for standard test methods is suggested. Investigation of influence of storage water on strength properties showed no difference for different types of water.—A. E. BEITLICH

Lime and magnesia determination in portland cement raw mix. G. W. JORDAN. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 62-63.—Objections to usual acid-alkali method are: Iceland spar used as standard may contain as much as 1½ per cent impurities; color change of phenolphthalein covers a pH range from 8.4 to 10.2; $Al(OH)_3$ is soluble in excess of NaOH and $Mg(OH)_2$ is hydrolized during the continuation of the titration for magnesia alone. These errors neutralize one another but it does not follow that the net result is correct. The author recommends using solutions exactly 2/5 N HCl and 1/5 N NaOH, standardizing on pure (Bureau of Standards) potassium-acid phthalate, as an alkametric standard, with phenolphthalein as an indicator. One gram of raw mix is dissolved in a 200 cc. graduated flask with 50 cc. of 2/5 N HCl, the flask attached to condenser and boiled. While still hot enough to exclude CO_2 excess acid is titrated with the 1/5 N NaOH, methyl red being used as indicator. 0.05 cc. is added to NaOH reading for difference in pH value of methyl red and phenolphthalein as indicators. Corrected reading is subtracted from 100 to give lime equivalents of lime and magnesium carbonates. The solution is then brought to a boil and 50 cc. of N/2 neutral sodium oxylate solution is added to precipitate lime. The whole is diluted to 200 cc. and filtered. 100 cc. of filtrate, which should be alkaline to methyl red but neutral to phenolphthalein is titrated with a dilute alcohol-glycerine solution of potassium palmitate which is just alkaline to phenolphthalein. Red end point with phenolphthalein shows when all magnesia is precipitated, 0.3 or 0.4 cc. being subtracted. This is amount necessary to give a red color with a blank solution just alkaline to methyl red. One cc. N/10 potassium palmitate equals 0.002 grams MgO. No more time is needed than for the ordinary method and author believes results will be much more accurate.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part IV. GEOFFREY MARTIN. *Rock Products*, Sept. 27, 1930, V. 33, No. 20, p. 53-55.—The author shows how he derives the following formula for kiln output, which he believes to be original, $M = 1.263 \frac{S}{V} d^2$ tons per hour, M being output, V the volume of exit gases per lb. of clinker made and S their terminal speed and d the diameter of the kiln. For good practice, M equals $0.1123d^2$ tons per hour. For unusually good practice M equals $0.153\ d^2$ tons per hour. These are with wet process kilns. With dry process kilns M should equal $0.1356\ d^2$. While the author was with the British Association of Portland Cement Manufacturers the work of a number of kilns was compared with what it should be by this formula. Improvements in practice were thus introduced that gave large

savings, in one case amounting to $75,000 a year. It appears from this formula that kiln production may be increased either by reducing the volume, V, of gas per lb. of clinker or by increasing its terminal speed, S. Hence reducing fuel consumption per lb. of clinker or reducing exit temperature will give a larger output. The author has patented the use of fans for increasing the terminal speed S of exit gases. Where it is desirable that the speed should be lower in certain parts of the kiln baffles are placed in these parts.—EDMUND SHAW

Researches on the rotary kiln in cement manufacture. Part V. GEOFFREY MARTIN. *Rock Products*, Oct. 11, 1930, V. 33, No. 21, p. 58-59.—To make clinker raw materials must be heated to 1371 C. To attain this temperature kiln walls must be heated to a higher temperature but not high enough to fuse brick work lining. Assuming that all heat necessary to make clinker passes through clinkering zone, then surface area of clinkering zone must radiate away absolutely as much heat as passes through the clinkering zone per unit of time. From the above, the formula for surface of kiln in clinkering zone is derived: $S = 315.7 \frac{M}{y}$ sq. ft. where M is the output in long tons of clinker per hour and y is pounds of clinker produced per pound of fuel. In an example y is assumed to be 3, corresponding to a fuel consumption of 33 per cent and required surface area works out to 105.2 M sq. ft. A table appended gives the surface areas of clinkering zones for outputs of 1 to 10 tons per hour. To find required length, L, of the clinkering zone: It has been shown that diameter of kiln, d, is $2.98\sqrt{M}$, and since $L = S/\pi d$, L works out to $11.24\sqrt{M}$. From these the size of the clinkering zone for any output of a rotary kiln may be calculated. A table is appended giving dimensions for outputs from 1 to 45 tons per hr. and for diameters from 3 ft. to 20 ft.—EDMUND SHAW

Constitution of cements. NORBERTO P. COSTA. *Doctoral Thesis*, National University of Buenos Aires (Argentina), 1930, 32 p.—Author presents a detailed microscopical study of three classes of cements. Clinkers or unpulverized cement were submitted to different methods of cooling after having been heated in an oven to 2597° F. for portland cement clinkers and to 2732° F. for aluminous cements. The methods of cooling were in the oven, in air, in sand, in oil, in water and in ice. Plates for examination were made by placing pieces of clinker into hot Canadian balsam. After cooling, these plates were ground and polished. Argentina portland cement obtained by dry process, one high early strength German portland cement obtained by wet process, and one French cement, high early strength, were used. Work is based on a study by Doctor Hendrick. Some of objects of thesis were to study the very great influence of large crystals in the clinker, its size, its grouping around a nucleus and its effect on the strength of cements. These characteristics were attributed to two fundamental operations in making a good cement, namely, heating and cooling. Conclusions by author are briefly: That a clinkerized cement is not simple mixture of aluminates and silicates, but it is very probable that, in its constitution, they form partly solid solutions of isomorphic mixtures of silicates of calcium and of iron, with aluminates of calcium. That the fused cements are formed by a mixture of aluminates, silicates and silica-aluminates of calcium, mixed with slag of silicates of iron. Aluminates seem to crystalize in the body of the slag, while silicates and silica-aluminates crystalize in the vitreous mass (vidrio) of their own composition. Predominant element in these cements is $5CaO, 3Al_2O_3$ in its unstable form. Portland cement obtained by wet process has same proportion of constituents as portland cement obtained by dry process, but in the first case most of the crystals of Alit and Celit are uniform and of somewhat larger size. Grouping of grains of Celit around a nucleus of Alit were observed. This was not observed in cements from the dry process. (Celit—solid solution of di-calcium aluminate in di-calcium silicate. Alit—solid solution of tri-calcium aluminate and tri-calcium silicate.) In

portland cement of high initial strength—obtained by the dry process—it is observed that proportion of Celit is greater than in other cements and grains of Celit and Alit are larger. Therefore, high strengths of a cement are a consequence of crystalline groupings, and of size of crystals, above all those of Alit. Most interesting form is grouping of crystals around a nucleus of other crystals, in a radiated spheroidal form. In portland cements, quenching, or cooling, causes a variation in some physical characteristics of crystals, but its chemical constitution remains constant. However, in fused cements, action of cooling changes chemical constitution. Many micro photographs and a two-page bibliography are included.—N. H. ROY

Investigations on protection of concrete against aggressive solutions. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Austrian tests on concrete columns with high quality steel reinforcements. (See ENGINEERING DESIGN—MISCELLANEOUS.)

PROPERTIES OF CONCRETE

The analysis of the solid concrete body, a means for production of concrete mixtures of great value. ERICH WEISE. *Die Bautechnik* (Germany), May 2 and 16, 1930, V. 8, No. 19-21, p. 284-7, 315-8.—Cementing properties, voids, cement content, water-cement ratio and compressive strength of concrete mixtures were studied and subjected to mathematical calculations. It is pointed out, that excess of water must be avoided. Decisive for properties of finished concrete is water content, which fact is expressed by w/c-ratio. Amount of water which adheres to aggregates is of great importance. Above a certain limit, an excess of water will be of less influence, when used for cast concrete. A number of formulas for calculation of valuable concrete mixtures are derived.—A. E. BEITLICH

Research comes out of the attic. WYATT BRUMMITT. *Concrete*, Dec., 1930, V. 37, No. 6, p. 28-30.—Industry today has taken over duties of research with same motive as that of struggling scientist of last generation. Portland Cement Association laboratory at Chicago is typical of industrial research. Work was begun with no definite goal but recognition of need for larger knowledge of cement and concrete. One of first projects was establishment of suitable measuring stick to gauge phases of work. Strength assumed of vital importance and effort made to determine effect of cement content, quantity and proportion of aggregate, and mixing time. Water content considered secondary until experiments led to water-cement ratio law. Three classes of investigations now interest researchers: Continued analysis and refinement of cement-producing process; perfected technique of using concrete under "impossible" conditions; and widening range of uses. Department, headed by F. R. McMillan, occupies half of 5 story building, employs 50 men, and maintains fellowship for research at Washington. Findings of quarter-million experiments are on file.—C. BACHMANN

The question concerning the water impermeability of concrete. VOGELER. *Bautechnik* (Germany), May, 1930, V. 8, No. 23, p. 342-3.—Author discusses directions, which are given in book: Specifications for mortars and concrete (Anweisung fuer Moertel und Beton; cf. *Zement* (Germany), Jan., 1930, V. 19, No. 3, p. 64, *J. of the Amer. Concr. Inst.*, April, 1930, V. 1, No. 6, Abstr. Sect. p. 68) concerning relations between aggregate gradation, mixing water and water permeability of concrete. Ostendorf gives his experiences with the preparation of concrete of high water impermeability and outlines further investigations in this line.—A. E. BEITLICH

Collapse of a lock in Huentel (Germany), investigation and repairs. RUDOLF JEHN. *Bautechnik* (Germany), June 27 and July 18, 1930, V. 8, No. 28-31, p. 438-41, 478-81.—Investigation of concrete and local conditions revealed that sewer water and ground water contained only small amounts of harmful sulphates and magnesium salts but very great amounts of aggressive carbon dioxide, which fact, together with porosity of concrete led to its destruction. Bottom slab of concrete showed stratification and layers of sound and disintegrated concrete. Repairs of lock consisted of removal of old concrete bottom and replacement by new slab. Concrete with an addition of trass was used and was worked to a very dense structure. It was provided with heavy reinforcements.—A. E. BEITLICH

Analysis of complicated concrete mixes. J. L. HEINTZMAN. *Rock Products*, Nov. 8, 1930, V. 33, No. 23, p. 62-64.—Author says that his results obtained by methods given have been upheld in court in suits for infringement of Straub cinder block patent. After explanation of method for qualitative examination to determine coarse aggregate used and some other things a method is given for analyzing concrete of cement and cinders. The sample is digested with 50 cc. 10 per cent HCl and insoluble determined. As cinders are 95 per cent insoluble and cement is about 98 per cent soluble, the insoluble found divided by 95 will be practically the percentage of cinders. Where sand, cinders and cement make the concrete, insoluble found as before is dried and weighed. Then sand is separated by Thoulet's heavy solution in which cinders float while sand sinks. With concretes of cement sand and slag, some of slag is removed and analyzed for insoluble, soluble silica, and lime and apparent specific gravity and loss on ignition are determined. The proportion of slag may be figured from either soluble silica or lime in the concrete and also in the slag. A similar method is used with concrete of cement, sand and limestone, a sample of the limestone being removed and analyzed separately. The limestone content may be found by formula which includes the CO_2 contents of the concrete and the limestone. With concretes of unknown composition there must be a careful visual examination and the use of physical methods, which are described, in addition to chemical analysis.—EDMUND SHAW

Relation between durability of concrete and durability of aggregates. F. H. JACKSON. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Basic requirements for durability are: the use of sound and durable aggregates, and maximum resistance to the entrance of water. The aggregates play a double role since the range of sizes plays an important part in producing impermeability. The characteristics of aggregates which affect their durability in concrete, and the effects of other factors such as permeability, water absorption, etc., are not well understood, and thorough investigations are needed. Concerted laboratory studies looking toward the formulation of standard tests for durability, and field investigations to correlate laboratory and structural experience are urged. Outlines for research work are presented.—HIGHWAY RESEARCH BOARD

Special investigation—curing of concrete pavement slabs. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Progress report setting forth the facts established by study of the available data with respect to strength, surface scaling, and volume changes as affected by the various well known curing methods, together with the tentative conclusions drawn by the committee. The report will include complete summaries of the data collected by the committee.—HIGHWAY RESEARCH BOARD

Durability of concrete in highways and highway structures. C. H. SCHOLER. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—General discussion of the factors involved . . . Types of disintegration observed as evidence of a lack of durability . . . Report of laboratory

and field investigations of durability as affected by method of testing, size and shape of specimens, method of sampling, character of aggregate, quality of cement paste, permeability and absorption as a factor in disintegration, and effect of water-proof coatings. An accelerated freezing and thawing test is presented as a means of testing the resistance of concrete and concrete aggregates to weathering agencies.—HIGHWAY RESEARCH BOARD

Concrete dams in Norway. *Report*, Norwegian Society of Engineers, July, 1930, 272 p., Den Norske Ingenioersforening, Oslo (Norway).—To investigate effect of water action when under hydraulic pressure on dams of concrete, the Norwegian Society of Engineers in Oslo have made thorough examination of 88 of more important concrete dams in Norway, some of which are more than 30 years old. These investigations were started in the summer 1926, and this is first report by committee. It was found that very few of the dams were uninjured and that injuries were not evenly distributed over dams, thus showing that proportions of concrete mixture must have varied during their construction. Joints had most of the injuries. Organic matter, iron and manganese substances were accumulated on the injured parts. Though none of the dams was found to be very seriously damaged, committee advised repair of injuries. Main reason for injuries is said to be chemical action of water. Thus water will penetrate cracks caused by temperature changes and frost and start to remove soluble parts of concrete. Flood water is known to be more or less acid and is therefore lime dissolving. Concrete mixture was found in most cases to be too lean, and it is pointed out that sooner or later a concrete structure under hydraulic pressure would be completely destroyed by this acid water. Use of membranous water proofing is suggested for protection against temperature cracks but no means has been found to protect this layer against mechanical action of water. Interesting suggestion proposes placing thin watertight concrete slab in front of dam, to make concrete dam dry by keeping air space between slab and dam. This type construction was used for Ringedal dam, 100 ft. high. Slab was 8 in. thick at top and 18½ in. thick at bottom, supported by columns with spiral reinforcing. Air space between slab and dam was 6.6 ft. Horizontal spacing of columns was 7½ ft. and vertical spacing varied from 10 ft. to 6 ft. at bottom.—ODD ALBERT

Investigations on protection of concrete against aggressive solutions. OTTO GRAF. *Zement* (Germany), Oct. 2, 9, 16, 30 and Nov. 6, 1930, V. 19, No. 40-41-42-44-45, p. 936-41, 970-4, 995-8, 1041-3, 1066-8.—Best protection of concrete is obtained, when aggressive waters are kept away from concrete. This fact is very often not sufficiently taken into consideration although it may be possible by certain changes of design of structure. Special shaping of structure may also lead to same effect. Running water has in general greater effect on concrete than standing water. When concrete is only partly immersed in aggressive water, dividing line between water and air is subjected to greatest corrosive effect. Water is raised in concrete body by capillary attraction, and salt contents in upper portion is increased by evaporation. Studies were made in order to find cements most resistant against sulphate waters, waters with great contents of uncombined carbon dioxide, certain salt solutions and organic media. Iron portland cement with low lime content loses less strength than normal portland cement when immersed in linseed oil. After storage in 10 per cent solution of magnesium sulphate, iron portland cement lost in 15½ months 18 per cent of its 28-day strength, while high early strength portland cement and blast furnace slag cement showed gains from 1 to 9 per cent. In magnesium chloride solution, cubes of high early strength cement were destroyed, iron portland cement lost 33 per cent and blast furnace slag cement gained 6-7 per cent in strength. Degree of fineness is very important: finer cements show less disintegration than coarser ones. Various cements were tested in different mixtures with sand and different consistencies and stored under magnesium sulphate and magnesium chloride solutions. Further sub-

ject of investigation was effect of cement content and gauging water on resistance against movement of liquids in concrete body. Graduation was studied which gives greatest resistance against chemical attack. Mixtures of 1:3 cement:sand were more resistant than 1:6 mixtures. Amount of mixing water should not exceed absolutely necessary limit. Presence of 3 per cent clay in sand caused great losses in strength when specimens were placed in aggressive solutions after a seven day moist closet storage, but retrogression did not take place when specimens were dried for 35 days before submitting to test. Additions of trass seem to improve concrete resistance. Finally influence of storage and finishing on concrete durability were studied.—A. E. BEITLICH

Fire damage in Riga. ED WEISS. *Beton Eisen* (Germany), Oct. 5, 1930, V. 29, No. 19, p. 344-348.—Buildings 134, 135 and 218 of Prowodnik Rubber Company, leased to the Soviet Government for flax storage suffered severe fire damage on April 11, 1930. Units 134 and 135 are five-story reinforced concrete structures, each three bays wide and fifty-eight bays long, measuring 92 ft. x 874 ft. in plan. Unit 218, a one-story structure with steel roof trusses, and measuring 99 ft. x 874 ft. in plan occupies the 99 ft. space separating these two buildings. Structures were erected in 1913 through cooperation of German and Swiss construction company. There is no indication of faulty construction but there is much to criticise in design. Beams and columns were evidently figured as non-continuous units with no regard for frame action—usual practice in Russia at that time. These units were tied together at points with small amount of steel and with small haunches. Design would not have been permitted according to Russian standards of 1914. Long spans and light sections give impression of weakness. Temperature joints divide structures into lengths of approximately 130 ft. It is surprising that wind forces for which no provision was made have had no visible effect throughout the 16 years during which buildings have been exposed to severe sea-storms. There were no fire-walls in any of three structures. At time of fire, flax to a depth of 10 ft. with open aisles along the columns and exterior walls, was stored on lower 3 floors of Unit 134, in Unit 218 and on ground floor and part of second floor of Unit 135. Fire was combatted by 30 hoses and 10 fire boats. An effort was made to flood the flax but to avoid striking hot concrete with cold stream. Steel beams, with lower surfaces exposed for attachment of shafting, reached white heat and concrete was heated in some cases to red glow. Columns in burning stories displayed a progressive outward bowing until collapse of structure. A deafening rattle like machine gun fire, perhaps due to failure of reinforcement, immediately preceded collapse. A total length of 620 ft. of Unit 134 collapsed after estimated exposure to fire of from 4 to 6 hrs. Portions still standing seem to indicate that cement was burned out of the concrete but that sand and gravel suffered no damage. In many cases column steel bulged and concrete scaled off, often for considerable depth into the core. The horizontal cracking and outward bowing evident in exterior columns at floor levels may be due largely to fact that these columns were comparatively fixed against lateral movement, at the foundation and at upper stories (where no flax was stored). Heating of slabs to 1110° F. and to 1830° F. would lead to lateral expansion of 4.5 in. and 7.1 in. respectively, at intermediate floor levels. These lateral forces would be sufficient to crack columns and to force them outwards. Destructive effect of a stream of water on hot concrete is shown by a photograph in which path of stream is indicated by a deep gouge in slab soffit and entire removal of concrete for one foot length of beam. The 9500 short tons of flax lost are valued at $1,750,000 while building damage is $250,000. Some insurance experts have expressed opinion based on this fire, that steel covered with brick or tile is more fireproof than reinforced concrete. Structure itself contradicts this conclusion for lean concrete used in upturned portion of spandrels broke as a unit at limits of various pours but showed little surface damage. A few brick walls on ground floor were completely destroyed. Mortar was burned out and the bricks burst.—A. A. BRIELMAIER

Small movements in concrete. (See ENGINEERING DESIGN—MISCELLANEOUS.)

ENGINEERING DESIGN

BRIDGES

Pneumatic caisson foundations for Japanese bridge. J. MASAKE. *Eng. News Record*, Nov. 27, 1930, V. 105, No. 22, p. 848-850.—A handsome concrete-arch bridge with stone facing has been built to replace old wooden bridge across Shinano River at Niigata, a city on north side of main island of Japan. The bridge consists of six spans of hingeless concrete arches, two each of 154 ft., 151 ft., and 142 ft.; in addition, there are two 48-ft. two-hinged land spans. A headroom of 18½ at low water is sufficient for the craft on the river. The riverbed is composed of layers of fine and coarse sand to depth of 80 ft. Pneumatic caissons were used for piers and abutments of main bridge. Sand from caissons was used in making concrete, along with river gravel and Japanese portland cement.—D. E. LARSON

Vierendeel bridges. E. BALIS. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from translation in *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 632-34.—In Belgium the Vierendeel girder is widely used and shows an economy of about 20 per cent over ordinary steel lattice structures, though when reinforced concrete is used the saving is not so great. The actual stresses in these bridges are about 6 per cent less than theoretical stresses. A closer agreement would require laborious calculations. Two classes of this type of bridge are in general use: (1) the bowstring and (2) the arch with monolithic decking.—JOSEPH MARIN

Reinforced concrete bridge at Wisbech. *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 609-12.—This reinforced concrete bridge now in the course of construction over the river Neme, at Wisbech, will be the largest span portal type bridge in Great Britian. Clear span will be 92 ft. 6 in. and width between parapets will be 45 ft. Choice of suitable design was influenced by considerable rise and fall of tides at bridge site. The maximum rise in tide is 15 ft. The type of construction adopted consisted of six main girders ranging in depth from 3 ft. 9½ in. to 3 ft. 7 in. with deep curved haunches at the ends. Width of beams varied from 26 in. to 37 in. Details of bridge and method of construction are given.—JOSEPH MARIN

Design of arched bridges with fixed supports. H. CARPENTER. *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 611-23.—Object is to develop a simplified method of design for arches with fixed supports. Tables are given for calculating stresses in parabolic, open and filled spandrel arches, with different rise-span ratios and varying moment of inertia. Simplification in the calculations is obtained by designing ring to coincide with line of pressure for dead load. In this way an economical design is obtained and stresses due to arch shortening are easily computed. Expressions are derived for horizontal thrust due to dead load and arch shortening, and for bending moments due to arch shortening both at crown and springing. Values of these thrusts and moments are given in tables for different ratios of the moment of inertia at the crown and springing, and for different rise-span ratios. A table is given for each of the three types of fixed arches considered. Similar tables to the above are given for changes due to temperature, giving the values of horizontal thrust and bending moments at crown and springing produced by changes in temperature.—JOSEPH MARIN

BUILDINGS

A new garage construction in Frankfurt a. M. (Germany). H. CRAEMER. *Zement* (Germany), Nov. 6, 1930, V. 19, No. 45, p. 1068-9.—Building of taxicab company is made entirely of reinforced concrete in modernistic style. Floor constructions of special design show interesting features. Details of columns, construction of windows and arrangement of storage and repair shops are illustrated.—A. E. BEITLICH

An example of the mechanical solution of a statically indeterminate structure. H. W. COULTAS. *Structural Eng.* (England), Oct., 1930, V. 8, No. 10, p. 344-366.—A complete analysis by means of Begg's Deformeter Apparatus is given for a six story frame two bays in width. Method of test and reduction of data for full size structure are given.—V. P. JENSEN

Static consideration of concrete mantle of supports in steel skeleton structures. LEOPOLD BERGER. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1018-20.—When steel columns of large skeleton structures are provided with concrete mantles it protects them against rust and also increases supporting strength. Great savings can be obtained by decreasing dimensions of steel column and reinforcing concrete with longitudinal irons. This combination is especially applicable to supporting columns of basements and lower floors. Example is given and calculation shows savings of 45 per cent of construction costs.—A. E. BEITLICH

Combining steel and concrete framing to effect economy. *Eng. News Record*, Nov. 27, 1930, V. 105, No. 22, p. 844-845.—An unusual combination of three types of structural framing in the 27-story Buckingham Building, 59 East Van Buren St., Chicago, includes built up steel columns of two different types, with both steel framing and concrete-beam framing, and also reinforced concrete columns with concrete-beam framing. Purpose of this combination was partly to effect economy and partly to keep column sizes within prescribed limits. Built up steel columns of H-section are used as far as the eleventh floor. Above the eleventh floor interior columns are also of steel, but of the Gray type, to the eighteenth floor, above which are spirally reinforced concrete columns. With columns varying in design at different heights, special methods were required for splicing the different types of steel and concrete columns. Typical details of these connections are shown. A two-way concrete floor of tile and joist type is another special feature of this building, and is used with both steel and concrete framing. Joists in both directions form a cellular construction with hollow tile filling in the rectangular spaces. Flat pieces of tile are placed under the concrete joists, being supported by dovetailed grooves in the filler tiles, so there is continuous tile surface for ceiling plastering. The floor slab over filler tiles is 2 in. thick and depth of tile is 4 in. Advantages of this type of floor construction are shallow depth and low dead load.—D. E. LARSON

Rigid rectangular frame foundation for Albany Telephone Building. G. W. GLICK. *Eng. News Record*, Nov. 27, 1930, V. 105, No. 22, p. 836-838.—A reinforced concrete foundation mat and two superimposed basement stories of reinforced concrete designed to act together as a rigid frame structure form interesting and unusual foundation for new section "B" of main telephone building at Albany, N. Y. This building recently completed by the New York Telephone Co. as extension to its existing plant, is 94 by 167 ft. with 3 basements and 11 stories of super-structure. Two factors made the foundation design complicated and important. These were: (1) character of the soil; and (2) necessity, because of delicate mechanism of telephone exchanges, that the building be free from unequal settlement. Building rests on clay formation over 100 ft. deep. Other buildings on this soil have settled over a long period

of time before finally coming to rest as the clay became stable under new load condition. This fact, with considerable economy in cost, led to choice of a continuous foundation mat which could distribute weight of building over its entire area. The substructure is advantageously combined into a rigid-frame, two-story-and-mat structure acting as a unit to resist unequal settlement, variations in loads and the undesirable center sag. Such a frame is virtually a truss without diagonal members and is known as the Vierendeel truss. To provide for expected settlement of building, all floors were set 1½ in. higher than those of adjoining building. Inspection of new building made in February, 1930, showed no settlement cracks in substructure or superstructure.—D. E. LARSON

Fire damage in Riga. (See PROPERTIES OF CONCRETE.)

DAMS

Stresses in gravity dams by principle of least work. B. F. JAKOBSEN. *Proc. Am. Soc. C. E.*, Sept., 1930, V. 56, No. 7, p. 1613.—Data are cited from English tests on model dams which show a non-linear stress distribution on horizontal planes. When the principle of least work was substituted for the usual assumption of a linear stress distribution in a theoretical investigation, a stress curve similar to that found in the English tests resulted. The proposed theory gives a higher bending stress at up-stream face and lower at down-stream face. It is believed that the principle of least work method will more nearly approach the true stresses than will results obtained by usual method. Belief is supported by explanations and mathematical reasoning. Previous works of Vogt, Cain, and others are cited frequently. The whole discussion applies to the masonry dam, which is usually made of concrete.—H. J. GILKEY

Circular concrete arch dam designed by simple method. R. P. V. MARQUARDSEN, *Concrete*, Nov., 1930, V. 37, No. 5, p. 33-38.—This is the second article on this method. The design formulas were stated dogmatically in October issue of *Concrete*. Present paper contains the derivations of the various formulas. The cylinder formula is used. Total compressive stress on any cross section of a hollow-cylinder wall is expressed. For purpose of analysis, the arch is assumed to be cut at the crown and to move due to radial loading, temperature and shrinkage stresses, and settlement of supports. Equations for these stresses are derived. Arch is then treated as a unit, fixed at both ends, and subjected to other stresses in addition to above mentioned ones. Bending moments, angle changes and horizontal movements are discussed and equations derived. Unit fiber stress for combined direct and bending action is derived and stated in a simple equation.—N. H. ROY

Blasting a precast dam into place. C. P. DUNN. *Civil Eng.*, Dec., 1930, V. 1, No. 3, p. 159-164.—One of most interesting and difficult features in building of a 200,000 kw. hydro-electric plant at Chute à Caron on the Saguenay River in northwestern Quebec was the diverting of swift stream by building on land a heavy reinforced concrete dam and dumping it bodily into place. Essential features involved in this scheme were (1) a fixed pier, relatively massive, to carry greater part of weight and thrust; (2) a small pier carrying part of weight, to be blasted away; (3) a cylindrical rolling face on fixed pier so designed and placed that precast dam would fall into an accurately predetermined position. Standing on end, the dam was 92 ft. high, 45 ft. thick up and downstream, and had a maximum depth of 40 ft., the bottom being contoured to fit river bottom as dam fell into place. Reinforcing consisted largely of old steel cables. Upon blasting the small pier away, the dam fell accurately into place and was not shattered by impact although a few small hairlike cracks appeared.—D. E. LARSON

The Boulder Canyon project. *Eng. Con.*, Nov., 1930, V. 37, No. 5, p. 398.—The Hoover Dam, of gravity-arch type, will rise about 727 ft. above foundation rock and raise water surface 582 ft. Length of crest will be about 950 ft., width at base 650 ft., and at top, 45 ft. River will be diverted during construction by means of coffer dam and four 50-ft. diameter concrete lined tunnels through the canyon walls. Only openings through the dam will be eight metal-lined conduits 6 ft. 9 in. in diameter for carrying the low flow of the river when the diversion tunnel plugs are being poured. The spillway will be two glory-hole type spillways discharging through 50-ft. diameter vertical shafts into the outside diversion tunnel. Tentative plans call for a power plant to develop one million horsepower. The All-American Canal will be 75 miles long and the Coachella branch 115 miles long. The dam and power plant will require 4,510,000 cu. yd. of concrete masonry. Five and one-half million bbl. of cement and nineteen million pounds of reinforcing steel will be required —N. H. ROY

The buttressed dam of uniform strength. HERMAN SCHORER. *Proc. Am. Soc. C. E.*, Nov., 1930, V. 56, No. 9, p. 1947.—The buttressed concrete dam, of which Ambursen and multiple-arch are best known illustrations, consists of three distinct members: deck, buttress, and foundation. The buttress is generally most important item as regards quantity of concrete. This has been designed on basis of vertical stress components and horizontal shearing stresses. It is proposed to substitute for buttresses a series of arched columns designed on basis of principle stresses. To eliminate trial computations, water pressure acting on an elementary deck area is considered to be transmitted to foundation in most direct manner by elementary arched columns. These elementary columns can then be combined into a monolithic buttress of uniform strength or they may be separated from one another by continuous joints to eliminate secondary stresses due to shrinkage, temperature changes, etc. Old method has sometimes been unsafe since consideration of shears on the horizontal section failed to take into account important components of stress. Proposed method offers economy and rationalizes design. The paper outlines mathematical analysis of method and gives references to detailed derivations and more complete manuscript on file in the Engineering Societies Library, New York.—H. J. GILKEY

Concrete dams in Norway. (See PROPERTIES OF CONCRETE.)

MISCELLANEOUS

Economical bases for stanchions—"Controvertist." *Structural Eng.* (England), Nov., 1930, V. 8, No. 11, p. 402-405.—For a steel column supported on a concrete base, the pressures on the sides of base have been included in computing the resistance of base to overturning moment due to wind pressure. Connection of column to base is considered for conditions where column is embedded in base and where column rests on top of the base.—V. P. JENSEN

Economic dimensions and shaping of slab girders of reinforced concrete under consideration of their own weight. ALPHONS CHMIELOWIEC. *Oesterr. Ing. Arch. Ver.* (Austria), Oct. 10, 1930, V. 82, No. 41-42, p. 360-1.—Work which was done by Mayer, Barck and Saliger on calculation of reinforced concrete structures is discussed. Based on mathematical considerations, a way is shown for better economic designs of slab girders in reinforced concrete construction practice.—A. E. BEITLICH

Reinforced concrete design simplified. JAMES R. GRIFFITH. *Concrete*, Nov., 1930, V. 37, No. 5, p. 31-32.—Chart 1 gives depth of concrete slab and area of steel necessary for balanced reinforcing. The factors employed are: span in feet, moment coefficient, live load, slab thickness and steel area. An

allowable steel stress of 20,000 lb. per sq. in., concrete stress of 1200 lb. per sq. in., and n equal to 10 are used. This chart may also be used for obtaining steel area for a slab thicker than is warranted for span and load conditions, using the steel required for the lesser conditions although in a slab thicker than necessary. This approximation is on the side of safety.—N. H. ROY

Austrian tests on concrete columns with high quality steel reinforcements. R. SALIGER. *Oesterr. Ing. Arch. Ver.* (Austria), Oct. 10, 1930, V. 82, No. 41-42, p. 361-2.—In connection with a comprehensive investigation on concrete columns with different reinforcements, conducted at laboratory for testing materials in Berlin-Dahlem (Germany) and at technical testing laboratory in Vienna (Austria), a number of column tests with high quality steels were made. Cross-section of steel varied from 4 to 14 per cent of total column cross-section. Columns were 9.8 ft. and 4 ft. long. Bearing strength was surprisingly high, and necessary cross-section of concrete column was not greater than cross-section of a riveted or welded steel column of same strength. Further tests are in preparation.—A. E. BEITLICH

Solving rectangular concrete members subjected to bending and direct stress. L. C. MAUGH. *Eng. News Record*, Nov. 6, 1930, V. 105, No. 19, p. 723-724.—The problem of designing rectangular reinforced concrete sections for bending and direct stress is now frequently encountered in use of rigid frames. In this article, author presents formulas and a diagram for design of such members. Numerical operations are made in same sequence as those for ordinary beams. This method is more direct than the methods usually presented in textbooks and the chances for numerical mistakes are less. Concrete is assumed to resist no tension and to have a constant modulus of elasticity.—D. E. LARSON

Folded plates (Faltwerke) and their application in reinforced concrete construction. H. CRAEMER. *Zement* (Germany), Oct. 16, 23, 1930, V. 19, No. 42-43, p. 990-2, 1020-2.—Author extends his studies concerning economic design and construction of reinforced concrete plates (c. f. *Zement*, July 24, 1930, V. 19, No. 30, p. 703-8; *J. Amer. Concr. Inst.*, Nov., 1930, V. 2, No. 3, Abstr. Sect. p. 72) on combination and arrangements of monolithic concrete plates junctions of which form certain angles. Analysis are given of stresses which occur in such combinations, and it is shown how this construction principle can be applied with great advantage to practice of silo and bin construction. Furthermore, consideration is given to construction of concrete roofs and multiple arrangements of this type. Writer refers to article by Ehlers in *Bauingenieur* (Germany), Feb. 21, 1930, V. 11, No. 8, p. 125-32 (*J. Amer. Concr. Inst.*, Nov., 1930, V. 2, No. 3, Abstr. Sect. p. 84) on a similar subject.—A. E. BEITLICH

Theory of similarity and models.* BENJAMIN F. GROAT. *Proc. Am. Soc. C. E.*, Oct., 1930, V. 55, No. 8, p. 1803.—Although Newton's principles of similarity and the theory of models were enunciated 250 years ago, the use of models by engineers is recent. Models often present the best form of preliminary analysis for constructing engineering works of great magnitude. The mathematical background that underlies the proper use of models is presented and attention is called to current fallacies. It is suggested that materials, including fluids should be sought, made, and arranged so as to adapt and make them available for service in models.

**Abstractor's Note:* It is deemed desirable to call the attention of the A. C. I. constituency to this paper because of the increasing use of models in connection with concrete design.—H. J. GILKEY

Concrete stadium for $6.60 per seat. *Eng. News Record*, Nov. 6, 1930, V. 105, No. 19, p. 721-722.—A concrete stadium has recently been designed for Sacramento Junior College at Sacramento, Calif., at a cost of $6.60 per

seat. By developing a design structurally independent of floorslab and using timber seats and walkways that could be put in place with a minimum of labor, cost of 22,000 seat structure was kept down to $145,000. The structural scheme consists essentially of concrete columns 14 in. square spaced 16 ft. apart both ways. These columns are connected at top by reinforced concrete girders carrying one intermediate beam each way, which reduces spans to 8 ft. With this span a 2 by 12-in. plank is ample for carrying maximum calculated load of 100 lb. per linear foot. Planks of this size were used both for seats and walkboards, the seat being stiffened by a 1 by 6-in. edge board.—D. E. LARSON

Grade separation structures untangle traffic jams. ROBERT C. KINGERY. *Concrete*, Nov., 1930, V. 37, No. 5, p. 17-19.—Four major types of structures or plans are employed in separation of intersecting highways. First type comprises two highways at different levels, with no means of turning from one highway to the other. In the second type there is an elevated structure which carries the central part of one of the highways over and across the full width of the other, all other pavements and the sidewalks on both roads remaining at grade. Third type is most desirable from the standpoint of abutting property owners. The central part of one highway is depressed, all other pavements and sidewalks remaining at grade. Fourth type is well suited to crossings where public property is available. One highway is partly depressed and other partly elevated. Diagonal lanes serve as connecting links between highways. Another effective arrangement which does not involve actual separation of grades is the traffic circle. It keeps traffic moving in both directions on both roads, although traffic on the circle itself is all in one direction, moving counter-clockwise.—C. BACHMANN

Analysis of continuous frames by distributing fixed-end moments. HARDY CROSS. *Proc. Am. Soc. C. E.*, May, 1930, V. 56, No. 5, p. 919-928.—While not restricted to concrete, the method of analysis is especially applicable to concrete construction since the method is intended to be an aid in obtaining the moments, shears and thrusts required in the design of complicated continuous frames from the reactions which can themselves be found by more or less standard means. The method is restricted by condition that joint must not be displaced but even this case can be indirectly treated. The writer emphasizes the fact that he is setting forth a *method of analysis* for the benefit of those who recognize that values of moments and shears cannot be found exactly but who desire a method of analysis that will combine reasonable precision with speed. The method is therefore one of successive approximations rather than an approximate method. Moreover it is a method rather than a treatise.—H. J. GILKEY

Small movements in concrete. W. H. GLANVILLE. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from *Concrete Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 634-43.—The paper deals with work of Building Research Station on small movements in concrete. From test data an empirical formula is obtained for the stress in reinforcing steel of concrete members due to shrinkage of concrete. Coefficients obtained from tests are given that are to be used in the empirical formula. Tests were made to determine the creep or flow of concrete under load for both plain and reinforced concrete. For plain concrete the effect of the following factors on flow of concrete was studied: (1) stress, (2) type of cement and age at loading, (3) cement content and (4) storage conditions. Tables are given for each of these factors giving the amount of creep. The tests show that (1) creep is approximately proportional to applied stress, (2) rapidity of hardening directly influences creep to considerable degree, (3) creep is approximately proportional to quantity of aggregate, and (4) influence of age of concrete at the time of loading is appreciable for about the first month.—JOSEPH MARIN

Nomogram for calculating the area of any number of bars of the same dimension. JORGE QUIJANO. *Ingenieria* (Mexico), May, 1930, V. 4, No. 5, p. 204.—A simple nomographic chart, one of a series, for use in reinforced concrete design, to compute areas of steel reinforcement. Left hand scale, dimension of bars, center scale area of bar and right hand scale number of bars, all in metric units. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating columns without spiral reinforcement (central load). JORGE QUIJANO. *Ingenieria* (Mexico), May, 1930, V. 4, No. 5, p. 209.—A simple alignment chart, one of a series, for use in reinforced concrete design, to solve formula $P = [A_c + (n - 1) A_s] f_c$. Metric units used throughout. Left hand scale A_c, middle scale P and right scale A_s for $n = 15$, $f_c = 500$ lb. per sq. in., $h/R < 40$. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the pressure of earth (or water). JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 227.—Alignment or nomographic chart for use in computing earth pressure for concrete retaining wall design by Rankine's formula. Explanation of use given on chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for economical design of T-beams. JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 237.—A nomographic chart, one of a series for use in reinforced concrete design and investigation, solving the relations $d = \sqrt{\frac{r\,M}{b^1 f_s}} + \frac{t}{2}$ and $M = A_s f_s (d - t/2)$. Left hand scale $(d - t/2)$, middle scales M/f_s and A_s and right hand scale b^1/r. Explanation of use given on full size chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the allowable distance between abutments of vertical or inclined reinforced concrete slabs as determined by diagonal tension and shear. JORGE QUIJANO. *Ingenieria* (Mexico), June, 1930, V. 4, No. 6, p. 253-254.—A nomographic chart for use in designing or investigating buttressed retaining walls which solves the relation $T = S (v - 2.8) b$. Explanation of use is given on a separate page. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating simple slabs and rectangular beams. JORGE QUIJANO. *Ingenieria* (Mexico), July, 1930, V. 4, No. 7, p. 280.—A simple alignment or nomographic chart, one of a series of such charts, for use in reinforced concrete design and investigation, solving the relations $d = 0.3558 \sqrt{M/b}$ and $A_s = 0.002532 \sqrt{Mb}$ for $n = 15$, $f_s = 18{,}000$ lb. per sq. in. and $f_c = 700$ lb. per sq. in. Left hand scale or line d, middle scales M and A_s, and right hand scale b. Explanation of use given on chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating slabs or beams with a certain degree of end restraint. JORGE QUIJANO. *Ingenieria* (Mexico), July, 1930, V. 4, No. 7, p. 291.—A simple alignment or nomographic chart, one of a series, for use in reinforced concrete design and investigation, solving the relation between A_s, d and M for $n = 15$, $f_s = 18{,}000$ lb. per sq. in. and $f_c = 700$ lb. per sq. in. Left hand scale or line L, middle scales A_s and d, and right hand scale W. Explanation of use given on chart. All in metric units.—C. G. CLAIR AND M. N. CLAIR

Nomograph for calculating slabs or beams with a certain degree of end restraint. JORGE QUIJANO. *Ingenieria* (Mexico), Sept., 1930, V. 4, No. 9, p. 342-343.—A simple alignment chart for use in reinforced concrete design solving the relation between A_s, d and M for $n = 15$, $f_s = 20{,}000$ lb. per sq. in.

and $f_c = 800$ lb. per sq. in. Left hand scale L, middle scales A, and d, and right hand scale W. Explanation of use given on chart and next page.—C. G. CLAIR AND M. N. CLAIR

Nomogram for determining the tension in the steel of simple T-beams. JORGE QUIJANO. *Ingenieria* (Mexico), April, 1930, V. 4, No. 4, p. 173.—A simple alignment chart, one of a series, for use in reinforced concrete design and investigation, solving the relation $M = 0.9\,dT$. Left hand scale T, center scale M and right scale d, all in metric units. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

Nomogram for calculating the moment due to the weight of a slab or beam. JORGE QUIJANO. *Ingenieria* (Mexico), April, 1930, V. 4, No. 4, p. 177.—A simple alignment chart, one of a series, for use in reinforced concrete design, solving the relation $M = 0.0003\,b\,(d + e)l.^2$ Left hand scale l, center scale M/b and right scale $(d + e)$, all in metric units. Explanation of use given on chart.—C. G. CLAIR AND M. N. CLAIR

ROADS AND PAVEMENTS

The design of a concrete mixture. J. E. MYERS AND MARK MORRIS. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Detailed description of steps involved in design of a mixture for concrete to be made of certain specific materials to have an average crushing strength of 4000 lb. per square inch at 28 days, to be of suitable consistency for pavement work, and to require the use of predetermined quantities of materials. The design was based upon methods proposed by Talbot and Richart with some variations. Previous experience with one of the materials used in this case was reported in the Seventh *Proceedings* of the Highway Research Board. Use of the designed mix on 15 miles of pavement, showed that the concrete was of satisfactory consistency, that the average strength was slightly in excess of the requirement, and that the cost of the materials used was 0.03 per cent less than the estimate.—HIGHWAY RESEARCH BOARD

Boulevard and Great Belgrano Park, planned for the City of Rosario. *Magazine Tecnico* (Argentina), June, 1930, p. 16-17.—Department of Navigation and Ports of City of Rosario, province of Buenos Aires, Argentina, proposes to construct avenue and park along Parana river. The avenue is to provide a proper view of river and the park to give proper setting for historical spot where the flag created for Belgrano first was unfurled. Two types of cross section are proposed for the avenue both occupying a total width of 164 ft. Type A cross section has two roadways 36 ft. wide, foot paths on each side of these roadways of 19 ft. 6 in. and 10 ft. 0 in. width respectively, and a central park or garden section 33 ft. wide. Type B cross section has two roadways 20 ft. 6 in. wide with footpaths of 19 ft. 6 in. on each side and a central park or garden section 45 ft. wide. Roadways are to be of reinforced concrete of 1:2:3 mix, that for the type A 7 in. thick with 9.2 lb. of steel per sq. yd. and for type B 6 in. thick with 9.2 lb. of steel per sq. yd. It is estimated that plan A would cost $1,425,-816.17 and plan B $1,087,204.60.—C. G. CLAIR AND M. N. CLAIR

FIELD CONSTRUCTION

BRIDGES

Bridges and roads of the nation (Argentina). *Magazine Tecnico* (Argentina), March, 1930, p. 15-26.—A review of progress during 1929 on various projects with details. Numerous bridges of reinforced concrete have been completed or are in the process of construction. The bridges are generally of reinforced concrete arch type with some of beam and girder. New road construction includes considerable concrete.—C. G. CLAIR AND M. N. CLAIR

Construction of Pancevo bridge over river Danube near Belgrad (Serbia). MAX HEINICKE. *Bautechnik* (Germany), June 6, 1930, V. 8, No. 24, p. 354-7.—Structure is truss bridge with 7 spans, which rest on 8 reinforced concrete piers. Approach on south side is reinforced concrete arch bridge with 5 spans. Length of truss bridge is about one mile, and of reinforced concrete arch bridge 393.7 ft. Distances between river piers from center to center are 528.2 and 531.5 ft. Spans of concrete bridge are about 83.7 ft. River piers are founded on layer of very hard clay 91.8 ft. below normal water level. Caisson method was used for their foundation; caissons are 95.1 ft. long, 41.0 ft. wide and 24.6 ft. high. Usually two piers were placed at same time, two travelling cranes being used for placing caissons and for transportation of concrete. Concrete was mixed in one-yard mixer which, together with aggregate silo and crane, was mounted on a barge. Piers and foundation slabs were made of 1:8 mix and top slabs of 1:4 mix. Parts of piers exposed to heavy ice pressures are lined with layer of natural stones. Depth of foundations of piers for reinforced concrete bridge is determined by depth of clay layer. One pier was placed using compressed air method, and two were made in open excavation with steel sheet piling of Larrsen type.—A. E. BEITLICH

The bridge over the marsh and river Warthe near Fichtwerder (Germany). KARL BERNHARD. *Bautechnik* (Germany), May 16 and 30, 1930, V. 8, No. 21-23, p. 311-5, 338-42.—Highway bridge over marsh and river consists of two parts: one steel structure of 459.3 ft. length, which crosses river at right angle and rests on three concrete river piers, and one reinforced concrete bridge over marsh, which is 1653.6 ft. long, resting on 25 piers. Concrete bridge approaches river bridge at slight angle and artificial island was built at their junction. Center opening in river is 295.3 ft. wide while two side openings are 82 ft. each. Piers of concrete bridge are placed in alternating distances of 78.1 and 82.0 ft. Roadway rests on 3 main longitudinal flat slab girders, spaced 7.4 ft. apart. Height of girders is 4.8 ft. Several drawings show details of their construction. River piers were built using compressed air method and consist of foundation 13.1 ft. wide protected against aggressive waters by brick wall, and upper shaft, 8.2 ft. wide. Foundation of piers of concrete bridge rests on concrete piles of 15.7 in. diameter. They were driven about 6 ft. into sound underground. Twelve piles support one pier. Article is accompanied by 35 illustrations which explain details of structure and portray all stages of construction.—A. E. BEITLICH

The reconstruction of the Niddatal bridge near Assenheim in Oberhessen (Germany). METZIG. *Bautechnik* (Germany), Aug. 22, 1930, V. 8, No. 36, p. 540-4.—Old railroad bridge of steel construction was rebuilt and provided with 5 reinforced concrete piers to replace 8 old steel structures. New piers were placed on new foundations. Excavations were protected by steel sheet pilings, 26.5 and 45.9 ft. long, of Larrsen type, driven from 16.4 to 23 ft. deep into sound underground. Bridge has two openings of 1490 ft. at each side and two openings of 1863 ft. between three middle piers. Foundations of piers consist of reinforced concrete slabs, 49.2 by 70.2, 43.9 by 70.2, 38.7 by 70.2 and 25.6 by 70.2 ft. Special care was required for one end pier foundation, which is directly by side and below foundation of old pier. Concreting was carried out in two sections which are tied together by heavy reinforcements. No protective coatings were necessary, since chemical analysis of groundwater showed absence of aggressive agents. Gravel was delivered by railroad cars and dumped from bridge through long wooden chutes (to prevent separation of different sizes) to mixer. Total of 157 cu. yards gravel was unloaded daily. Mixer had capacity of 17½ cu. ft. Construction period lasted five months.—A. E. BEITLICH

Concrete pits of bascule bridge resist high water pressure. *Concrete*, Nov., 1930, V. 37, No. 5, p. 13-16.—Concrete in the walls and floors of machinery pits at both abutments of the double leaf, Wabash Ave. bridge, at

Chicago resists a pressure head of 25 ft. of water. Pits comprise space formed by the four walls of the abutments. Floor is concrete slab 5 ft. thick and the entire structure acts as a unit supported on concrete sub-piers or caissons. Sub-piers are carried down 75 ft. below datum, the base of each pier being belled out. Joint between walls of pit and floor slab was double keyed and surface was roughened. Concrete in pit walls was placed in vertical stages varying from 4 ft. at bottom to 8 ft. at top. Each stage was concreted without interruption, and joints between stages were double keyed and roughened. Torpedo sand and gravel were used as fine and coarse aggregate. For sub-piers and south machinery pit, standard portland cement was used. For north pit, which was first to be concreted, Super cement was used, concrete being placed during cold weather. South pier was concreted during warmer weather. Proportions in north pit were 1:2.5:3.5, with 6 gals. water per sack of cement. For south pit and all sub-piers, mix was 1:1.7:3.2, with 6 gals. water per sack of cement. Slump in all cases was 4 in. Concrete mixing plants for pits were separate installations, because presence of river navigation made transportation across stream inadvisable. From mixing plant for north pit which included mixer, aggregate measuring hoppers for volume measurement of aggregate and water control equipment, concrete was delivered into a hopper and wheeled in buggies over runways.—C. BACHMANN

BUILDINGS

Light-concrete for construction of 400 dwelling houses in Berlin-Zehlendorf (Germany). WEISS. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1090-2.—Construction was carried out with a concrete containing gravel and Rhenish pumice stones as aggregates. Heat insulating properties make this an ideal material for small dwelling buildings. Interesting and economic methods were used for putting up formwork and distributing of concrete with a minimum of laborers. About 455 cu. yds. of concrete were placed daily. Outside of buildings was finished with white cement.—A. E. BEITLICH

Five scattered hospital buildings concreted from one plant. *Concrete*, Dec., 1930, V. 37, No. 6, p. 17-18.—On government hospital project at Northport, Long Island, N. Y., comprising six reinforced concrete buildings, concrete for five buildings was mixed at centrally located plant equipped with batchers, water-measuring device and 10-S mixer, located directly between two of larger buildings. For two buildings concrete was distributed by buggies, direct from hopper alongside main hoist tower, and above mixing plant. Hoisting tower and long chute sloped about 35 deg. with horizontal was used for building farther away. Plastic consistency was maintained by addition of gallon of admixture to each 2-sack batch. Mixing time was 1 min., 10 sec. Proportions specified and used for columns and footings and for floor slabs, beams and girders, were 1:2:4 with 6½ gal. of water per sack of cement. For roadway and plain concrete work, mix was 1:2½:5 with 7½ gal. of water per sack of cement. Mixing plant was set low as practicable and trucks hauling aggregate ran up incline built with excavated material and dumped directly into bins through open side near top of mixing plant.—C. BACHMANN

Construction of new city hall in Ruestringen (Germany). HANS KUBALL. *Bauingenieur* (Germany), May 16, 1930, V. 11, No. 20, p. 349-52.—Structure consists of three parts, separated by expansion joints; a 15-story tower structure and two side buildings. Underground conditions and presence of aggressive agents in ground water made foundation on wooden piles necessary. About 1000 piles, 12 in. and 14 in. in diameter and 40 ft. long, were driven. Great care was taken in protecting concrete against action of water. Very rich cement mortar (1:2) with addition of waterproofing compounds was used for cementing a protecting layer of bricks. Detailed description is given

of several interesting floor constructions. High early strength cement was used for tower structure and normal portland cement for construction of side buildings.—A. E. BEITLICH

How to save in concrete form work. A. B. MACMILLAN. *Concrete*, Nov., 1930, V. 37, No. 5, p. 27-29.—Details of form work around steel beams and in intervening floor panels of building having steel frame encased in concrete are shown in illustration. The entire floor form work is supported on short cross pieces 3 in. by 4 in. about 27 in. apart, held in place against the beam bottom by wire passed over steel beam and tightened by twisting above top flange of beam. Wires pass through form work at edges of beam bottom, with beam sides set against them and supported on cross-pieces. Form work for floor construction is assembled by tacking beam bottom plank to cross-pieces, then wiring this assembly to steel beams after small concrete blocks are inserted between beam bottom plank and lower flange of steel beam. Width of beam bottom must be 4 in. more than width of bottom flange of steel beam, leaving 2 in. space for concrete to pass on each side of flange. Next the lower boards of beam sides are set in place, then 2-in. planks that serve as blocking for floor joists are set on cross-pieces, and lower boards of beam sides tacked lightly against them. Floor form panels are placed and upper boards of beam sides are set and tacked lightly to ends of a few joists. Floor panels have previously been assembled by nailing ⅞-in. boards to joists. If enough men and hoisting equipment are available, a floor form panel may be made up to full area of floor panel. If lighter form units are necessary, several smaller form panels may be used instead of a large one. The ⅞-in. boards of floor panel project over beam sides. Removal of this form will be made easier if floor boards adjacent to beam are omitted from assembled panel, and set in place and lightly tacked after floor panel is set. When the form work is removed, floor panel can be lowered as soon as plank blocking is taken away. To remove floor form construction: Shore up one edge of floor form panel by temporary frame; cut wires at beam where floor panel is shored, and remove cross-pieces and beam bottom. Remove 2-in. blocking and lower boards of beam sides. Let down edge of floor panel by lowering temporary shoring frame, and remove panel and upper boards of beam sides. Ends of wires protruding from concrete may be cut off flush with the concrete unless needed to support lath and plaster. Form work for lens sidewalks consists essentially of boxes supported on ledges, beam sides being slightly battered to facilitate removal. Form work for window sill consists essentially of yokes shaped like inverted "U." Tin strap at upper right-hand corner is to provide partial hinge action so yoke can be released. Form for parapet coping involves keeping true alignment for long stretches of wall. Use continuous liners in combination with cleats and spreaders all of 3 in. by 4 in. material. Staggering liners with respect to those on opposite side of coping helps maintain alignment.—C. BACHMANN

DAMS

Construction of the locks in the river Main upstream Aschaffenburg (Germany). FRIEDEL. *Zement* (Germany), Oct. 23, 1930, V. 19, No. 43, p. 1015-8.—Progress is reported of construction of 13 locks in river Main which enable navigation and serve for power production. Each lock has three openings which are 114.8 ft. wide. Ship locks are 984.3 ft. long and 39.7 ft. wide allowing passage of 3 or 4 ships at a time. On opposite side of river are located the hydro power stations. Two turbines in each station transform energy of 4,600 cu. ft. of water per second to 21 millions K. W. hours. For each station will be consumed about 80 to 90,000 cu. yd. of concrete. Locks at Obernau (Germany) are completed. Details of its dimensions and operation of gates are given. Locks at Kleinwallstadt, Klingenberg-Trennfurt and Kleinheubach are still under construction and work at Freudenberg was just started. Gradual development of these jobs is clearly illustrated.—A. E. BEITLICH

The power stations of the Lake Schluchsee. ADOLF EISENLOHR. *Bautechnik* (Germany), April 18, 1930, V. 8, No. 17, p. 259-63.—Water, which runs from Lake Schluchsee (Germany) and several other points of southern part of Black Forest (Germany) to river Rhine is utilized in three hydro power stations in Haeusern, Witznau and Waldshut. Difference between levels of Schluchsee and Rhine is 2950 ft. Meteorologic observations during 30 year period showed high average of rainfall in this region, 39.4 to 70.9 in. Steps between power stations are 659.5, 918.6 and 360.9 ft. Upper station in Haeusern is now under construction. Several small waters are collected and led to Lake Schluchsee through concrete pipe which is 5.6 ft. in diameter. Dam of gravity type, which is 108.3 ft. high and crest of which is 872.7 ft. long, dams water to level 95.1 ft. higher than original one, forming a lake of 1.9 sq. miles surface with a capacity of 141 million cu. yds. of water. Overflow is located in middle of dam. Dam is straight and has triangular cross-section. Concrete pressure line between dam and power station shows very interesting design. Inlet is located about 46 ft. below lake level. Pipe 3.7 miles long leads in nearly straight line to power station in Haeusern and has circular cross-section with inner diameter of 13.4 ft. Two gates are provided for regulation of water stream. Further description is given of construction of foundation of power house with its 4 turbines. Water basin for second hydro power station is formed by dam 495.4 ft. long, which has very favorable location. It is slightly curved and of same cross-section as first structure. Its pressure line is 1.7 miles long and 16.4 ft. in diameter. Arrangement of sites of construction, transportation and preparation of structural materials is described and illustrated.—A. E. BEITLICH

Blasting a pre-cast dam into place. (See ENGINEERING DESIGN—DAMS.)

MISCELLANEOUS

Construction cranes. "Baukrane." R. CAJAR. 1930, R. Oldenbourg, Munich and Berlin (Germany), 275 p., 354 illustrations, R. M. 18.00. Reviewed in *Z. Oesterr. Ing. Arch. Ver.* (Austria), Oct. 10, 1930, V. 82, No. 41-42, p. 367.—Book is comprehensive treatise on subject of construction cranes. Besides detailed description of different kinds of cranes, their operation and power consumption, several calculations are given for steel construction of such cranes and steel ropes.—A. E. BEITLICH

Methods of obtaining watertight concrete. WILLI SERKIN. *Betonwerk* (Germany), June 1, 1930, V. 18, No. 30.—Watertight concrete is dependent on non-absorbent aggregate, dense mortar, the sand grains of which are so graded to produce the least voids. Methods of attaining this grading are analyzed. The cement should be further made water repellent with an admixture prepared for this purpose.—H. FRAUENFELDER

Lectures on reinforced concrete construction. "Vorlesungen uber Eisenbeton." E. PROBST. 1929, Julius Springer, Berlin, V. 2, 2nd edition, 540 pp., 61 illustrations, RM. 31.50. Reviewed in *Bauingenieur* (Germany), Jan. 24, 1930, V. 11, No. 4, p. 69.—In four main chapters following subjects are discussed: reinforced concrete construction of superstructures, bridges, marine structures and miscellaneous structures. It deals mostly with practical application of different construction methods for large halls, silos, ceilings, foundations, different types of concrete bridges, sewers, pressure pipes, locks, dams, breakwaters, caissons, wells, poles and chimneys. V. 1, which appeared in 1923 illustrates theoretical side of subject.—A. E. BEITLICH

Concrete piles. M. J. MCCARTHY. *Reports*, First International Congress for Concrete and Reinforced Concrete, Liege, 1930. Abstracted from *Conc. Constr. Eng.* (England), Nov., 1930, V. 25, No. 11, p. 628-32.—Methods of

casting concrete piles are given for both piles that are cast in molds before being driven into the ground and for piles cast in place. Advantages of piles cast in place over the pre-cast piles are given. Pre-cast piles are most suitable for jetties, piers, and wharves. Five methods are given for casting piles in place and the methods are compared.—JOSEPH MARIN

Coast protection. *Conc. Building Conc. Products* (England), Oct., 1930, V. 5, No. 10, p. 139.—For protection of a rocky coast with concrete, a 12 in. thickness of 1:2:4 concrete is recommended, reinforced with ½-in. dia. bars at 12 in. centers each way. The bars should project 9 in. into the rock and 9 in. into the concrete, with both ends hooked.—JOHN E. ADAMS

The concrete pump, a new concreting method. HEIDORN. *Der Bauingenieur* (Germany), May 30, 1930, V. 11, No. 22, p. 381-4.—Experiences with a new concrete pump during construction of "German House" in Flensburg (Germany) and several other construction jobs are described. Concrete was pumped 88.6 ft. high. Output was 350 cu. ft. per hr. Control tests showed, that concrete delivered from pump has great uniformity in composition and strength. Special pipe construction facilitates mounting and cleaning. Transportation and operation costs are very low.—A. E. BEITLICH

New York subway construction—IV. *Eng. News Record*, Nov. 20, 1930, V. 105, No. 21, p. 796-800.—Precast-concrete blocks are being used for lining shield-driven subway tunnels from Prospect Park to Prospect Ave. and Terrace Place in Brooklyn. The use of precast-block or monolithic concrete, at option of contractor, was specified as an alternate to cast-iron segments to determine whether a lining could be secured that would be cheaper than, but as desirable as, cast-iron lining for shield driven tunnels. On tangents the lining consists of rings of 10 standard blocks, each block 6 ft. 1 in. long on chord of outside arc, 18 in. thick and 30 in. wide, forming a tube 16 ft. 9 in. inside diameter and 19 ft. 9 in. outside diameter. Each block contains 0.8 cu. yd. of concrete and weighs 3,200 lb. Curve alignment is taken care of by tapered rings, narrowing by 3.4 in., 1⅜ in., or 2¾ in., in precisely the same manner as cast-iron taper rings. Using these three different taper rings in combination with standard rings in proper proportion gives any desired curvature of tunnel. —D. E. LARSON

New locks in Ijmuiden (Holland). ANTON VAN RINSUM. *Die Bautechnik* (Germany), May 9, 1930, V. 8, No. 20, p. 305-8.—Structure is one of greatest European ship locks. Its inner dimensions are: 1312.4 ft. long, 164.1 ft. wide and 49.2 ft. deep. Ground water level was lowered 26.2 ft. and excavation was carried 42.7 ft. below normal sea water level. Further depth of 85.3 ft. was reached by using Larssen steel sheet piling 88 ft. long driven 3 ft. in hard layer of clay. Foundation rests on concrete piles made at site of construction. A total of 6100 piles were driven. Bottom slab of lock consumed 14,130 cu. ft. of concrete and was placed in 18 hours without interruption. About 31,780 cu. ft. of concrete were placed in 48 hrs. for each side wall. Concrete mixture was very rich and made with an addition of trass as protection against sea water action. Dune sand and blast furnace slag cement without trass addition were used for part of structure. Lock heads consumed about 2½ million cu. ft. concrete of which 250,000 cu. ft. were used for concrete piles. Concrete was poured with plastic consistency, 820 lbs. of cement for 35 cu. ft. of concrete. Concrete mixer furnished 1,060 cu. ft. concrete in one hour. One distribution tower of 255 ft. high was erected.—A. E. BEITLICH

Reinforced concrete construction. Part I. Raw materials and their production. "Der Eisenbetonbau. Teil I. Rohstoffe und Herstellung." O. HENKEL. 1929, H. A. Ludwig Degener, Leipzig (Germany), 112 p., 136 illustrations, R. M. 4.00. Reviewed in *Z. Oesterr. Ing. Arch. Ver.* (Austria), Oct. 24, 1930, V. 82, No. 43-44, p. 379.—Ways and methods of reinforced

concrete investigations and research in modern institutes for testing structural materials are clearly illustrated. Concrete construction control is discussed in all details. Engines, mixers, means of transportation, storage and preparation of aggregates and cement are described.—A. E. BEITLICH

Novel shield and hexagonal concrete liner blocks features of tunnel job. *Eng. News Record,* Nov. 13, 1930, V. 105, No. 20, p. 771-773.—An electrical conduit tunnel, 4 ft. in diameter and 100 ft. long, involving the use of a new design of interlocking hexagonal precast concrete lining blocks and a shield having several innovations, has just been driven for the Philadelphia Electric Co., in North Philadelphia. Tunnel has an inside diameter of 4 ft. and block lining thickness of 5 in. One complete ring of lining contains 18 blocks, but because of their hexagonal shape 9 alternate segments are always half a block width ahead of the other nine. This feature was an essential part of shield operation. Half of the 18 shield jacks were bearing against blocks in place, while the alternate nine, retracted, were extended one at a time under low pressure to hold blocks of next half ring in place as they were set. All of segments are uniform, no key blocks being used. The blocks are elongated octahedrons, 5 in. thick (radial to center line of tunnel), 8 in. wide (longitudinal to center line of tunnel), formed by two hexagonal faces, the outer face curved to form outer diameter of tunnel, and inner face composed of three flat planes. They are fully interlocking, 3 of contact faces having grooves $\frac{3}{16}$ in. deep and other 3 having tongues $\frac{1}{2}$ in. high. Cost of this type of construction is said to compare favorably with open-cut construction up to 10 ft. deep and to be cheaper for greater depths.—D. E. LARSON

Movable distribution arrangements for cast concrete. V. MENG. *Zement* (Germany), Nov. 13, 1930, V. 19, No. 46, p. 1094-6.—Modern concrete distributor which was used during construction of double locks at Kembs i. E. (France) of ship channel between Kehl and Basle (Switzerland), includes one tower located at each side of excavation and movable on 4 tracks. Height of each tower is 196.8 ft. and it can distribute concrete over a radius of 230 ft. Widths of tracks are 6.6, 59.1 and 6.6 ft. Total weight without ballast is 275 tons; traveling speed is 8.2 ft. per minute. At base of tower are two concrete mixers of $1\frac{3}{4}$ cu. yd. capacity, automatic measuring devices for water and concrete silo for 5 cu. yd. of concrete. Elevator delivers material to second silo on tower from which it is further distributed by means of pipes. Each tower can place approximately 78 cu. yd. concrete per hour, but this figure can easily be increased by installing $2\frac{1}{2}$ cu. yd. mixers.—A. E. BEITLICH

Construction of a new breakwater in the harbour of Stralsund (Germany). SEETZEN. *Bautechnik* (Germany), June 27, 1930, V. 8, No. 28, p. 435-8.—Article describes construction of three breakwaters, which are 1263.1, 1780.5 and 367.5 ft. long and replace old wooden pile structures. Greater part of new breakwaters is reinforced concrete construction. Great difficulties were encountered in finding sound underground for foundations.—A. E. BEITLICH

Ready-mixed concrete sold on strength basis at low unit cost. *Concrete,* Dec., 1930, V. 37, No. 6, p. 21-22.—Mechanical installations wherever feasible make low unit costs possible for Brooklyn plant. Cement company barge transports bulk cement in railway freight cars. Five-in. pump conveys cement to three main storage bins, from which series of screw conveyors carry cement horizontally to receiving hopper of vertical bucket elevator. Latter raises cement to 600 bbl., 2-compartment service bin, for standard and special cements, located near mixer. Self-cleaning, 300 ton aggregate bin above mixer is filled by stiff-leg derrick from nearby barges. Ten by 10-ft. hopper fitted with spout and mounted on turntable is located on top of aggregate bin. Proportioning equipment attached to aggregate bin includes quadruple, 16,000 lb. weighing batcher having four radial gates for bin and four-beam scale of

multiple type. Other attachments are 3,000-lb. weighing batcher for cement and 250-gal. water-cement ratio measuring tank. Proportioning units are adequate for 3-yd. batch. Controls and scales arranged so one man can measure cement, water, both aggregates, and discharge materials into mixer and operate mixer from operating platform. Mixer is timed for 1½ minute intervals between batches. Time required for discharge and charging is 15 seconds. Delivery trucks have 5 cu. yd. capacity, full load being made up by 3 yd. and 2 yd. batches. Field testing equipment is installed on mixer floor. Commercial laboratory representative checks control of strength, takes cylinders for tests, makes changes due to moisture variations. Concrete is usually sold on strength basis; proportions generally are 1:2:4; 1:2½:5; 1:3:6. Forty-five trucks with agitator bodies deliver up to 800 cu. yd. in 9-hour day. Aggregate bins are lined with cylinder heaters using steam under pressure of 25 to 30 lbs. Operating force includes 8 men.—C. BACHMANN

Ship locks of the Shannon power station in Ireland. FRIEDRICH WIENER. *Die Bautechnik* (Germany), May 30, 1930, V. 8, No. 23, p. 335-8.—Lock consists of two sections founded on solid rocks. Two side walls of upper section rest on 3-ft. thick reinforced concrete slab made of special cement and basalt aggregates. Bottom of lower section lies 47.4 ft. below that of upper section. Construction of it was similar. Box shaped gates of reinforced concrete are operated from bridges, spanning lock. General description of construction details and of operation of lock is given. (cf. M. Enzweiler, *Die Bautechnik* (Germany), June, 1930, V. 8, No. 25, p. 359-70, *J. of the Amer. Concr. Inst.*, Oct., 1930, V. 2, No. 2, Abstr. Sect. p. 50).—A. E. BEITLICH

New drainage pumping stations of Emscher Corporation in Essen (Germany). ALEXANDER RAMSHORN. *Bautechnik* (Germany), May 9 and 23, 1930, V. 8, No. 20-22, p. 299-303, 330-2.—Extended drainage system with great pumping station in Karnap drains area in mining district subject to slow sinking. Site of pumping station was chosen where greatest sinking is expected to take place to insure sufficient gradient at all times. Station consists of underground pumping room and engine house. Underground structure is reinforced concrete frame construction carefully made tight against ground water. Frame is a twin-cell box frame of Ehler type. Walls are divided into strips 3 ft. wide, reinforced according to load. Ground below building can sink over one third of its length without endangering structure. Excavation was made inside steel sheet pilings. Greatest care was taken in making a very dense, watertight concrete. Aggregates were carefully examined and improved by sieving and proportioning according to Fuller curve. Blast furnace slag cement was used for preparation of a stiff, plastic concrete. Excavation was first provided with layer of cinders on which was placed 6-in. layer of concrete. Brick walls were erected on all sides with a ¼-in. layer of insulating material. Then reinforced concrete floor slab and walls were erected. Description includes details of arrangement of suction and pressure pipes, engine house and adjacent building. Construction of second pumping station in Sterkrade (Germany), built in 1929, of similar design is described.—A. E. BEITLICH

Remarkable concrete and reinforced concrete structures in the steel works of Friedrich Krupp A. G. in Essen-Borbeck (Germany) FRANZ SCHLUETER. *Zement* (Germany), Oct. 2-9, 1930, V. 19, No. 40-41, p. 943-8, 968-70.—Outstanding concrete and reinforced concrete structures of greatest German steel works are described. (1) Quay at south side of main harbor has total length of 755 ft. and is made of concrete mixed 1 part blast furnace slag cement and 6 parts of gravel with an addition of trass. Precautions were taken for raising quay 6 ft., since underground is expected to sink slowly on account of undermining. Two travelling cranes for unloading boats run on top of quay and on reinforced concrete wall 781 ft. long. Forms for this wall consisted of one movable section suspended from wooden travelling crane on which was

concrete distribution equipment. (2) Storage bins for iron ore and limestone consist of one structure 361 ft. long with total capacity of 13,000 cu. yds. It is divided into 24 sections, which are 18.0 ft. long for ores and 22.1 ft. long for limestone. Partition walls are of reinforced concrete. Floors have slope of 40 and 45 degrees. (3) Foundation of one blast furnace consists of reinforced concrete slab, 55.8 by 59.1 ft. and 11.5 ft. thick. It carries a cylindrical reinforced concrete structure, inner core of which is filled with lean concrete mixture, and 4 main columns of blast furnace. (4) Chimney 230 ft. high and 9.8 ft. inner diameter at top was constructed of hollow concrete bricks, made at site of construction and provided with vertical and circular reinforcements. At height of 105 ft. above floor is located water tank around stack, with capacity of 211,000 gallons. (5) Boiler and power house are made exclusively of concrete. Furthermore, following concrete structures are described: water pipe 850 ft. long and foundation of a gas tank of 39,000 cu. yd. capacity.—A. E. BEITLICH

Bonding Concrete. *Contr. J.* (England), Oct. 1, 1930, V. 103, No. 2677, p. 1033.—Evils attendant on careless bonding of new concrete to old are not always appreciated. Shrinkage stress resistance is reduced and a poorly constructed joint may become susceptible to moisture percolation. Disintegration from frost, penetration of chemicals in solution and disfiguring efflorescence may result. Surface of well matured concrete becomes very hard and unless crust is removed by chipping, brushing with wire brush, rinsing with clean water, and a neat cement slurry added, satisfactory bond will not be formed. After slurry application, cement mortar of similar composition to that embodied in new concrete, and of plastic consistency, should be applied and followed immediately by new concrete, which should be well worked against the joint. In laboratory, excellent results have been obtained by following slurry with cement mortar, composed of two parts cement to one part sand, by weight, and following this by mortar, 2 parts sand and 1 part cement by weight. When fresh concrete is placed against this differential shrinkage is reduced to a minimum. Emphasis is laid on need for removing laitance film and layer of porous concrete immediately below it. Forms should if possible be filled to a point slightly above required height, so that poorer material may be struck off before concrete stiffens up, and within less than four hours. Too wet mixes should be avoided and when depositing under water, concrete should not be allowed to fall through water. Hacking of surface for bonding should only be done when concrete is very hard. In this case similar mortar to that in the new concrete mix should be applied in a ½ in. layer, and new concrete worked up against it. (c. f. Bonding new concrete to old. Norman Davey. *Building Research Bul. No. 9*, England).—JOHN E. ADAMS

Monolithic reinforced concrete chimneys. OTTO HOFFMANN. *Zement* (Germany), Aug. 28, 1930, V. 19, No. 35, p. 828-34.—Historical review illustrates development of construction methods for tall chimneys. Concrete form bricks with inlaid vertical and ring reinforcements were used in former years. Very little was known about stresses, in tall chimneys due to uneven heating, nor about heat permeability and radiation. Theoretical and practical experiments led to a rapid development in monolithic construction methods with adjustable, sliding forms. Investigations by Moersch, Graf, Doering, Knoblauch, Koch and Kleinlogel are cited and discussed. Use of high early strength cement made rapid concreting possible. Several outstanding reinforced concrete chimney structures, are described which show clearly progress made during last years. Chimney, 196.85 ft. high, in Obercassel (Germany), shows application of sliding forms in their early stages. System "Heine," used for construction of stack for electro power plant in Finkenheerd (Germany), shows remarkable progress in steel forms. Inner and outer forms consist of steel rings 3 ft. high, supported by scaffold on top of structure.

Structural materials are elevated through interior. Chimney is 361 ft. high and 16.2 ft. in diameter at its top. Two tallest European reinforced concrete chimneys were erected in 1929 for great power plant near Schwandorf in Bavaria (Germany). They are about 460 ft. high with inner top diameters of 24.6 and 18 ft. Structures were completed in 5 months. Stack for chemical plant was erected during severe frost periods with temperatures as low as 13° F. Entire top of structure with scaffolds was protected by closed canvas tent and was heated with steam to about 50° F. Base rests on reinforced concrete foundation slab 45.3 ft. in diameter. Total height of chimney is 262.5 ft. and its diameter at top is 11.8 ft. Thickness of walls ranges from 4.7 to 9.8 in. A very dense concrete made of high early strength blast furnace slag cement was used. Stack is lined with acid resistant bricks, 9.8 in. thick which withstand action of SO^2 containing gases which enter stack at temperature of 392° F. A chimney, which will be 426.5 ft. high when finished, is under construction now. It has a second vertical stack in its interior 131.2 ft. high serving as a flue dust collector.—A. E. BEITLICH

Completion of Moffat Tunnel of Colorado. CLIFFORD ALLEN BETTS. *Proc. Am. Soc. C. E.*, April, 1930, V. 56, No. 4, p. 679-716.—Paper supplements a former paper by R. H. Keays (cf. "Construction Methods on the Moffat Tunnel," *Trans. Am. Soc. C. E.*, 1928, V. 92, p. 63) outlining progress to August 1925. Construction started in 1923 and tunnel was opened to traffic Feb. 26, 1928. Location is 50 mi. west of Denver, at elevation about 9,200 with length 6.2 miles (longest railroad tunnel in America when constructed). It is 2,800 ft. underground at continental divide and involved 750,000 cu. yd. excavation, 16,000,000-ft. board measure timber, 2,500,000-lb. dynamite, 700,000-lb. drill steel, 700 mi. drill holes, 28,000,000-kw.-hr. electrical energy and 25,000,000 man-hours labor. The work includes a large amount of massive concrete and gunited sections. Final net cost $15,470,000. Project involves in addition to standard 16 by 24 ft. railroad tunnel an 8 by 8-ft. pioneer bore from which the excavation at several different headings could proceed from 8 by 8-ft. crosscuts at about 1,500 ft. intervals. The pioneer bore is to be used as a water tunnel to augment Denver's water supply. West 2.5 mi. was through soft ground and heavy timbering, steel reinforcement, concrete lining, and development of "Lewis Traveling Cantilever Girder" were among important engineering features involved. Surveys and closures, housing facilities, geological and general engineering features are covered in the paper. Concrete mixers and guns for the job cost $26,000. Aggregates were crushed and screened from the tough granite excavated from eastern half of the tunnel. Some stretches of tunnel subject to air slacking were lined with a 1½ 3-in. layer of gunite (1:3 proportions) at about one tenth the cost of timbering. This proved very satisfactory when there was no weight to support. In bad ground very heavy concrete lining was required. Mix was 1:2:3 (water-cement ratio 0.8 to 1.0). Special reground cement was used to give strengths of 2,000 and 4,500 lb. per sq. in. in 48 hours and 7 days respectively. Reinforcement of 30-lb. rails and 1-in. bars was placed between 5½-ton steel sets. Folding wooden forms in 4 ft. sections were used for invert. Then steel forms mounted on a timber jumbo carriage were moved along tracks on finished invert and jacked and bolted into position. Side and top concrete was fed from concrete guns through 8-in. steel pipe to crown of arch. Flowing into place was assisted by workmen with shovels and one-man air hammers used outside the forms. Mixing time was 1½ min.; gun pressure 80 to 110 lb. per sq. in. and a slump of 6 to 7 in. at mixer was necessary to obtain a 4-in. slump in the form. Both stationary and moving gun setups were used and inspection windows were provided in forms. There was segregation in pipe lines if more than 100 ft. long. Concrete thickness ranged from 1 to 5 ft. and a total of 916-lin. ft. of steel lining was buried in 12,000 cu. yd. of concrete including that poured around wooden sets. Provision has been made for placing some added concrete lining each year if necessary. Concrete lining cost $270 per ft. (8.5 cu. yd. at $31.74). A length

of 1,485-ft. of the pioneer water tunnel (62 ft. away) was lined with corrugated pressed steel liner plates which were covered with 4-in. of gunite. This supported loads estimated at 129,000 lb. per lin. ft.—H. J. GILKEY

ROADS AND PAVEMENTS

Maintenance costs. H. K. BISHOP. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Description of some of the ramifications involved in attempting to compare maintenance costs in different jurisdictions, such as quality of maintenance, accounting methods, equipment, traffic, width and climate.—HIGHWAY RESEARCH BOARD

Repair of pavement settlement. W. H. ROOT. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—A description of a unique method for raising sections of pavement that have settled, by pumping mud through holes in the slabs, with operating and cost data.—HIGHWAY RESEARCH BOARD

Fillers and cushion courses for brick and block pavements. J. S. CRANDELL. *Summaries Papers Reports*, Highway Research Board, Dec., 1930. —The ideal filler to meet the conditions set forth in the report has not yet been found. Some experiments that are under way but not yet ready for study are mentioned.—HIGHWAY RESEARCH BOARD

Recent developments in construction methods and appliances. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—An illustrated discussion showing the latest types of all kinds of equipment peculiar to road construction, and tracing relationships between improved road work, lowered costs and machinery development. Improvements have been brought about principally through (1) changes made necessary by reason of modifications in construction practices, (2) the adaption of existing devices to new uses, and (3) refinements and improvements made primarily for the purpose of increasing operating efficiency. Attention is also called to some further improvements, the need for which is apparent.—HIGHWAY RESEARCH BOARD

Removal of ice from pavements. B. C. TINEY. *Summaries Papers Reports*, Highway Research Board, Dec., 1930.—Attention is called to the need for an investigation to develop information relative to the most effective and economical materials and methods for use in dealing with icy road surfaces.—HIGHWAY RESEARCH BOARD

Economies of grading and paving. A. STELLHORN. *Contr. J.* (England), Oct. 22, 1930, V. 103, No. 2680, p. 1268.—An excess thickness of slab is regarded as evidence of poor engineering supervision since it may represent considerable expense in a large job. It is suggested that screed of finishing machine and sub-grader be set absolutely dependent on each other, a section of grade being subgraded and checked for thickness by running finishing machine in front of mixer and testing grade. The extra work is much less in value than the cost of concrete saved on use of scratch board, which usually causes a grade to be $\frac{1}{8}$ to $\frac{3}{8}$ in. low. Some notes are given on the method of dealing with the excavation, haulage and refilling, rolling and finishing of grades, and then on the paving operations.—JOHN E. ADAMS

Systemization of Vercellese Street in the Province of Milano. ERIK SILVA. *Le Strade* (Italy), March, 1930, p. 79-84.—Development of provincial streets radiating from city of Milano is most important part of a general plan of street systemization which has been progressing since 1927, and includes not only placing of new road surfaces, but also of rearrangement of the gas, electric, sewer, telephone, power and car lines. Vercelles St., an important artery is 82 ft. wide constructed, after much study, with a central safety zone

23 ft., two thoroughfares of 19.7 ft. and two side walks of 9.8 ft. wide. The street sections are given a slope of 2 per cent to the right side for drainage and to give easier driving. The base of the road is concrete and the surface asphalt. On a section of 2390 sq. yd. "Walterite," a penetration type of bituminous surface, was used. Every effort was made to plan the road so that subsequent breaking through to change piping, etc., would not be necessary. The cost was about $24.50 per sq. yd. of which $10.70 was for paving.—C. G. CLAIR AND M. N. CLAIR

Modern concrete pavements. E. K. SMITH. *Canadian Eng.*, Sept. 9, 1930, V. 59, No. 11, p. 309.—Concrete pavements in cities built 36 years ago are still giving good service. In the United States 50,000,000 sq. yd. of concrete pavements are built yearly. Materials are tested before use and proper mixture is determined by laboratory studies. Some methods which have resulted in economy with increase in quality are proportioning aggregates by weight, central mixing plants with delivery to job in some type of agitator body, greater attention to limitation of water content of batches, rigid steel side forms which assist in obtaining a smooth surface free from waves or inequalities, improved finishing machines which permit the use of batches of low water content, greater attention to curing by means of "ponding" or the application of some surface coating which prevents evaporation, and proper placing of expansion joints properly protected by a mastic filler.—G. M. WILLIAMS

Seattle's concrete pavements cured with continuous sprinkler. H. F. FAULKNER. *Concrete*, Dec., 1930, V. 37, No. 6, p. 19-20.—City's specifications require that as soon as pavement can be sprinkled without injury to fresh concrete, it shall be covered with burlap sheets two feet wider than pavement, the contractor supplying sufficient burlap to cover day's run, with 2 ft. lap for each sheet. Covering must be kept continuously wet and maintained in place until system of continuous sprinklers is installed following morning. Sprinklers shall be operated day and night without interruption for 10 days. Perfected sprinkler head throws very fine spray or mist, and when screwed into tees on 1¼-in. pipeline about every 20 ft. will continuously water 25-ft. pavement. Automatic, intermittent valve control in hydrant lead supplying line of sprinklers is also permitted. Device may be adjusted to give on-and-off periods of sprinkling, saving 75 per cent of water. To overcome strong wind, line of heads is tipped toward wind.—C. BACHMANN

A section of pavement with rigid reinforcement. ERIK SILVA. *Le Strade* (Italy), Feb., 1930, p. 58-61.—Aurelio Saffi Street in Milano was paved in 1910 with compressed asphalt and has stood up well under traffic except for portion occupied by two car tracks. Street was repaired by excluding traffic and placing a rigid reinforced concrete base 12 in. thick under the rails. Rails are attached to short sections of cross beams spaced a foot or two apart and embedded in the concrete. Machine cut blocks of porphyry quartz with bituminous mortar joints were used for wearing surface. A sand cushion is used between concrete base and blocks. Street was opened to traffic after one month. The economics of this type of pavement relative to the less rigid types is discussed.—C. G. CLAIR AND M. N. CLAIR

Patching concrete roads. GEREKE. *Betonstrasse* (Germany), July, 1930, V. 5, No. 6.—Tests of repairing concrete roads on a test street in Braunschweig showed good results. One part (by volume) cement, 1.6 parts sharp sand, 2.4 parts granite chips, ¼ in. to 1 in., mixed with 1 part of Sika No. 3 and 1.5 parts water were used. Holes were first washed with a mix of one part sand and one part cement. Material was applied in 6 to 7 minutes. Tests were made after 45 hours and showed different results for different kinds of cement used. It was estimated that according to test results the patched places could be opened for traffic after 20 to 24 hr.—R. J. DIETRICH

Concrete pavements. *Magazine Tecnico* (Argentina), April, 1930, p. 23.—A general statement of advantages of concrete pavements as found from their use in Argentina. The status of concrete road construction in the Province of Buenos Aires, Argentina, as of September 1929, is given as:

	Sq. Yd.	Cost Per Sq. Yd.
Constructed	125,800	$15.20
In Construction	103,800	11.10
Authorized	210,000	10.70
Projects ready	70,500	13.45
Projects being studied	478,000	11.50

This gives a total of 990,000 square yards at a general average price of $11.90 per sq. yd.—C. G. CLAIR AND M. N. CLAIR

SHOP MANUFACTURE

Observations on cast stone and the cast stone industry. *Betonwerk*, (Germany), April 6, 1930, V. 18, No. 21.—The proper name for this product is discussed. "Betonwerkstein" appears most popular. The product and its qualifications are described.—H. FRAUENFELDER

Concrete structural members and concrete products in slab construction. R. ROLL. *Beton Eisen* (Germany), Oct. 20, 1930, V. 29, No. 20, p. 370-373.—A brief summary is given of types of precast reinforced concrete load-carrying members and of concrete blocks for use in slab construction. Details of the various systems are shown in the figures. The article presents a view of the present stage of this field in Germany, and companies engaged in the manufacture of these products are listed.—A. A. BRIELMAIER

Profitable manufacture of concrete building units. FRED A. SAGER. *Concrete*, Dec., 1930, V. 37, No. 6, p. 33.—Business of producing and selling concrete products exhibits widest range in methods adopted and used by various concerns engaged in such manufacture. If, by keeping simple records, small plant can change manufacturing schedules from a haphazard to an orderly one with resulting saving, course should be adopted. Fundamentals of good plant design should be known and, if not embodied in present layout, worked into it. Most suitable equipment should be used, though in some cases this may not mean the most efficient machine. This series of articles will analyze business of moderate sized plant and arrive at certain fundamental conclusions as to stock of all types to be carried; plant required; general features of economic plant design and use of plant land; use of simple records to interpret costs; matters of deliveries and contact with customer during delivery; and markets and sales.—C. BACHMANN

Concrete units win way into large buildings. RUSSELL ACKER. *Concrete*, Dec., 1930, V. 37, No. 6, p. 13-16.—In bidding for Fort Wayne, Indiana, masonry market on quality basis, efficient plant having large production volume was found necessary and equipment installed accordingly. Outstanding unit is automatic Anchor junior stripper with tamper, power feeder and power strikeoff, having daily capacity of 2,500 standard size units. Large variety of sizes and quick mold changes are possible. Brick-making attachment added recently produces brick economically with low capital investment. Other units are a Hobbs downface machine under an Anchor floor tamper, with a daily capacity of 500 faced, standard size units, and Anchor tamper over a stripper for making two light-weight concrete roofing slabs at one operation. Masonry units are transported to one of three kilns on cars and steam-cured not less than 12 hours, then moved to yard on transfer track. Four men, one at mixer, one machine man and off-bearer, one all-around man, and one on truck delivery, are employed. Three 1-ton trucks comprise delivery equipment. Calls on architects and other logical prospects met with little success until

samples were left at offices and use of concrete as partition units in large eastern and southern buildings cited. Laboratory tests, building code committee recognition of units, and finally inspections of the plant, with high-grade equipment producing clean-cut block, neatly stored, converted architects. Largest order has been for 100,000 units; others for 15,000 to 20,000.—C. BACHMANN

Concrete cable covers. *Conc. Building Conc. Products* (England), Oct., 1930, V. 5, No. 10, p. 133.—To replace earlier clay product for protection of underground cables, many firms are manufacturing concrete cable covers, which are less easily broken, and which are provided with interlocking devices. A description is given of the latest type of cover, which is 12 in. by 6 in. by 1½ or 2 in. thick and the ends of which are concave and convex. To ensure that these ends shall remain tightly together, covers are provided with loops of galvanized iron wire projecting at each end, the loop at concave end being passed through loop at convex end of adjacent cover, and turned over with the fingers. These wires pass through the cover and act as reinforcement as well as tie, and serve to hold the pieces together in event of their being struck by a pick. Tops of the covers are stamped "electricity." They are composed of clean shingle, and sand from ⅜ in. down mixed with portland cement in the proportions of 1:2:4 and the materials are consolidated in moulds by vibration. Other products such as curb sections and flags are described, with illustrations of plant.—JOHN E. ADAMS

New method of stone preservation. *Betonwerk* (Germany), Oct. 26, 1930, V. 18, No. 43.—It is stated that each kind of stone and cast stone requires special treatment for its preservation. The treatment must not prevent breathing of stone. Experiments have been conducted with Silicic acid dissolved in alcohol or benzol as a surface treatment. This surface treatment hydrolizes slowly and cements grains together. Research being continued.—H. FRAUENFELDER

Tightening the requirements for making pipe C. M. HOWARD, *Concrete Products*, Oct., 1930, V. 39, No. 4, p. 20-21.—In plotting results of a number of sieve analyses it was noticeable that material passing through No. 8 sieve was quite uniform, so this sieve was taken as basis for comparison. Samples were broken on No. 8 screen and percentage of each sample retained was noted. Then the portion passing No. 8 was screened and results plotted. Uniformity of material below screen was quite startling, as was varying percentage of total sample passing. The result is quite obvious. A great deal of variation in material can be eliminated by breaking on No. 8 screen. The aggregate should be controlled by using a maximum of ⅜-in. gravel in all thin-shelled pipe; and it is believed that this is a maximum to be used in any machine-tamped pipe regardless of the thickness, because tightness of compacting is a matter of how large a rock the tamp stick will bed into the dry mix required, and necessity for compactness is as great in one size of pipe as in another. Tamp stick clamp has to be tighter in proportion to effective end area of tamp stick. In large size pipe the stick is larger, friction on stick is set tighter, and there is just that much more chance that tamp stick will have to bed two or three of maximum size aggregate particles in same blow.—E. S. HANSON